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ELASTICITY AND STRENGTH

of

MATERIALS USED IN ENGINEERING CONSTRUCTION

SECTION V

Economic Theory of Steel Railroad Bridge Design

By

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PREFACE

The three-fold increase in the weight of railway locomotives and cars in the past thirty years has been accompanied by no progressive improvement in the triangulation of the truss frame to support this increased loading.

Investigation of the precise number and form of the triangular frames which should be combined in a truss to secure the maximum stiffness and minimum weight has been given no noticeable consideration heretofore from the theoretical standpoint.

The following discussion will be directed toward elucidation of the underlying principles of economic design and the application of those principles to a sufficient range of span lengths to show a curve, page 57, indicating the reduction of weight effected by their application.

The indicated reduction in cost of the ordinary railroad truss bridge is so great that it should appeal to those in charge of the improvement, betterment and maintenance of our railroads, particularly so because accompanied by a large increase in rigidity and correspondingly reduced maintenance and increased life of the bridge.

The reduction in cost through reduced weight is enlarged by the use of fewer joints, reduced field riveting, simplification of the joints by the uniformity of the sections, reduction of the number of different pieces to handle, thereby simplifying and reducing the cost of shop work and erection.

Economic bridge design is so far removed from an exact science that the ideas of what should be the weight of long simple spans varies between different engineers from 50 to 100 percent as indicated by the discussion Trans. Am. Soc. C. E. 1922, page 965, where Mr. Steinman states his careful estimate of a simple 775-foot span based on detail calculations of all sections under the A. R. E. A. specifications shows a weight per lineal foot of 23,000 pounds, while the writer estimates the weight of two similar spans at half that amount. Mr. Lindenthal in the same paper believes a two-span continuous truss saves 20 percent over two simple spans, whereas the author figures that the continuous truss spans are 40 percent heavier than two simple spans should be.

This divergence in weight is not to be wondered at when we consider the A. R. E. A. specification in its broader aspects, to wit: The depth of trusses shall not be less than one-tenth of the span, Art. 51. Art. 12, panel lengths shall not exceed 1½ times the width center to center of trusses. Now the economic proportions for even the 200-feet through span are 2½ times as great as the limitations of this specification.

Again, the quality of the steel in our A. R. E. A. specification has exhibited under test, lower compressive resistance than that of wrought iron in use thirty years ago and worse than that it has been found to fail suddenly in Howard's tests of full size columns. A better grade is called for under the specifications given in the appendix, in which a definite discard is required and the method of determining an excess over the tolerance limits of the deleterious elements, phosphorus and sulphur, is specified in such wise that the search will be conducted (if the excess exists) with a better chance of success than that for the proverbial needle in a hay stack. In this general specification, the rules of design are given so concisely under the heading of proportion of parts that a nine-page index will not be needed to find the requirements.

The difficulty with a too general specification, to wit, one that endeavors to specify a structure that may be either pin connected or riveted, lies in the fact that these different kinds of construction are not analogous and the mixture of antipodal requirements for the different types is productive of both confusion and waste.

A general analysis of one or two long span structures is presented in the closing chapter with an indication of what might have been accomplished had there been a broader investigation along practical lines to secure the most economic and rigid structure possible for the location in hand.

C. A. P. TURNER,
Minneapolis, August, 1924.

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CHAPTER XXIII

RAILROAD BRIDGES

Live Load, Impact, Dead Weight, Wind Strain and Aesthetic Design

1. Introductory: Preliminary to the design of any frame the service which it is to render, the live load and its manner of application must first be known or determined. Then the sections necessary to carry the live load may be approximated and from this weight the additional areas required by the structure to safely maintain its own weight can be approximately determined and the final design worked out.

The **raison d'etre** of the present forms of bridge structures may well be investigated from a historical review of the requirements of railroad bridges and the origin of present practice as a background for the critical analysis of their substantiality and economic merit on the one hand or imperfections on the other.

2. Historical: Commencing with the age of steel, thirty-five years ago, the average locomotive used on American railways was less than one hundred tons in weight. Thus, the old standards of the Edgemoor Bridge Works of that period show tabulations for Cooper's Class A consolidation locomotive weighing 85½ tons; his Class B was 80½ tons; his extra heavy A 104 tons; Wabash Railroad had an 88-ton typical engine loading; the Pennsylvania Railroad 90½ tons Class R; the New York Lake Erie & Western 91 tons; while the coal carrying roads, Lehigh Valley and Philadelphia Reading, had 125-ton engines.

As the mechanical departments of the railroads brought out newer and heavier engines, placing the drive wheels a few inches nearer together or one and two feet further apart, the laborious computation of moment diagrams for these minor variations in wheel loads specified was performed by each bridge company bidding on the work and the labor involved in the preparation of innumerable standards of this kind was enormous and a most uncalled for waste.

In 1894, Mr. Theodore Cooper proposed a compromise standard type of loading consisting of two consolidation engines coupled together with an E classification based on the numerical kip weight of the driving wheels. That is an E 30 Engine, weighing 106 tons, would have 30,000 pounds on each of the four pairs of drive wheels; and an E 40 would have 40,000 pounds, an E 50 would have 50,000 pounds. At the time this compromise standard was proposed an E 40 was presumed to be heavy enough to meet future requirements. Today E 60 has been adopted by the A. R. E. A. and some have proposed to increase the E 60 or substitute other and different generalized loadings.

In the accompanying cut reproduced from Mr. Steinman's paper in the Transactions of the American Society C. E., 1920, page 606, Cooper's E 60 loading is shown together with the heaviest Mallet or other type engines now in existence. The upper diagram shows the variation in maximum moments expressed in percentages of Cooper's E 60 loading. The lower two diagrams show the floor beam reactions and end shears plotted from the same base line.

The diagram shows that for spans over 100 feet the maximum moments exceed Cooper's E 60 but little; that for panel lengths of 40 feet and over the beam shears and end shears exceed those given by Cooper's loading but a small amount.

The divergence in moment between the Mallet type and the Cooper E generalization of locomotive loading lies in the bringing together of the groups of heavy wheel load concentrations rather than a material increase in the total maximum load for considerable span lengths.

Now such variation is material to shorter spans only as disclosed by the upper diagram. We will accordingly investigate this discrepancy by plotting a curve of equivalent uniform load with the Cooper Class E 60 and noting its peculiarities. Such a curve is shown in the accompanying figure on page 8.

For both the reactions and moments, it will be observed that the curve runs with substantial uniformity from 160 to 800 feet and it decreases according to a regular law from 10 feet down to 30 feet when there is a divergence from regularity because concentrations arranged as in the Cooper E loading cannot occur in such wise as to give a regular gradation of equivalent uniform loading with the increase in span. This divergence in the regularity of the curve of uniform loading has been rectified in the full line of the diagram, the dotted line showing the variations due to the particular spacing of the group of wheels relative to the span length. By correction of this irregularity an equivalent load standard is presented which with continuous stringers is ample for any present day loadings when the impact is given rational consideration.

3. Conventional Uniform Load is the treatment of a uniform load as tho concentrated at the panel points and the analysis of live load stress by this conventional method consists in applying

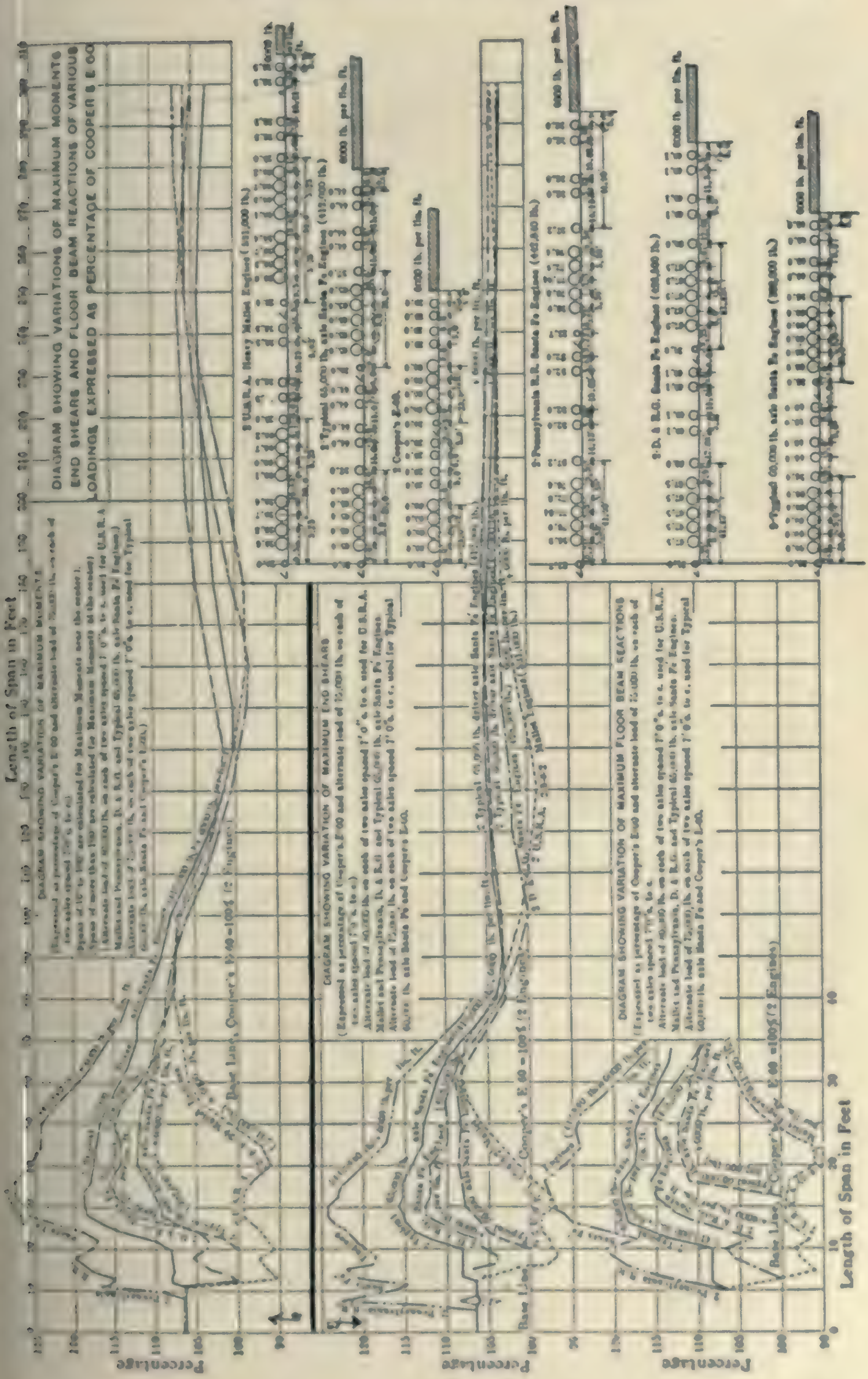


DIAGRAM SHOWING VARIATIONS OF MAXIMUM MOMENTS END SHEARS AND FLOOR BEAM REACTIONS OF VARIOUS LOADINGS EXPRESSED AS PERCENTAGE OF COOPER'S E 60
Length of Span in Feet
Percentage
DIAGRAM SHOWING VARIATION OF MAXIMUM MOMENTS
DIAGRAM SHOWING VARIATION OF MAXIMUM END SHEARS
DIAGRAM SHOWING VARIATION OF MAXIMUM FLOOR BEAM REACTIONS
Base Line, Cooper's E 60 = 100% (2 Engines)
U.S.R.A. Heavy Mallet Engines
Typical Santa Fe Engines
Cooper's E-60
Pennsylvania R.R. Santa Fe Engines
D. & R.G. Santa Fe Engines
6000 lb. per lin. ft.

this panel load consecutively on panel points from one end of the bridge to the other. This conventional method results in the same computed stresses for chord members as tho the load were treated by an exact method and a slight excess for web members on the side of safety. For the design of a camel back truss it permits the use of the simple coefficient shear diagram requiring check of only one member to substantiate the accuracy of all the coefficients graphically determined. Therefore, the method is ideal in its simplicity and numerous suggestions have been made for the application of this method to the railroad bridge truss in place of the complicated wheel diagram.

The unscientific manner of presenting the equivalent uniform load method is accountable for its lack of general adoption. Thus if an equivalent uniform load be determined for the chords for a given span length, the web members computed for this uniform load in the conventional manner will present stresses smaller than computed for the engine loading and because the lighter web members will be the first to be over-strained by an increase in loading this method was insufficient. But such is not a scientific method of application of the equivalent uniform load diagram.

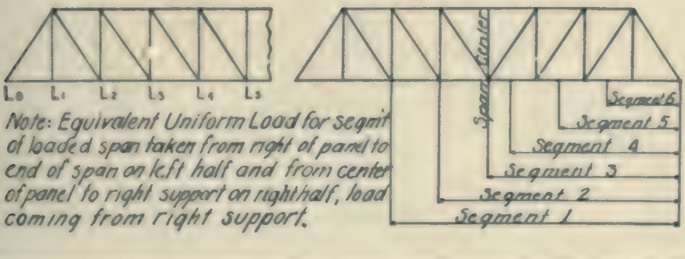
4. A Rational Equivalent Uniform Load may be determined from the curve of the diagram by taking that uniform load for computing the panel concentration for a span equaling the length of the loaded segment for the stresses in the web members. Then treating this load in the conventional manner the web stresses will agree closely with those for the wheel load diagram for trusses of five or more panels or exceed the web stresses for a smaller number of panels, so that it is always on the safe side, its divergence from the computed wheel load stresses is in general negligible and the work of computation may be performed in less than one-tenth the time with reduced chance of error because of the simplicity of the procedure.

Suppose for example we have a five panel 200-foot span. The live load on the first panel for web stress would be taken at 9,500 pounds per lineal foot from the diagram; when two panels are loaded there would be 80 feet of load and the uniform live load per lineal foot from which the panel concentrations are figured would be taken as 8,500 pounds from the *diagram. In like manner when three panels are considered fully loaded the panel concentrations would be figured at 8,000 pounds a foot. For 160 feet loaded it would be 7,500 pounds. The method gives a very close approxi-

*See Page 8 for diagram

mation to the engine loading because the shear at the left when the load comes on from the right is the moment of the loaded segment times its distance from the right abutment divided by the span length.

The following table for determining the shear due to equivalent uniform load is computed taking this uniform load as that for the segment of the span up to the center of the panel before the live load has passed the center of span and after passing the center up to the preceding panel point.


L0
L1
L2
L3
L4
L5
Note: Equivalent Uniform Load for segm't of loaded span taken from right of panel to end of span on left half and from center of panel to right support on right half, load coming from right support.
Span Center
Segment 6
Segment 5
Segment 4
Segment 3
Segment 2
Segment 1

No. of Panels	Panels	Exact	Equivalent Uniform Load	Exact	Equivalent Uniform Load	Exact	Equivalent Uniform Load
		26′	26′	30′	30′	35′	35′
4	L^1L^2 L^2L^3	73.5 22.0	88.2 25.6	81.9 24.3	95.6 41.2	....	119.5 45.2
5	L^1L^2 L^2L^3	120.1 56.3	127.0 70.5	136.8 63.4	143.0 75.5	155.4 72.9	161.6 87.6
6	L^1L^2 L^2L^3 L^3L^4	156.5 97.8 45.7	165.1 105.9 55.2	188.0 110.9 50.9	191.1 119.2 61.8	212.7 126.1 59.1	221.6 134.4 69.8
7	L^1L^2 L^2L^3 L^3L^4 L^4L^5	211.1 141.1 81.9 38.5	213.0 143.0 90.7 47.3	237.5 158.3 93.4 42.7	242.6 163.8 99.5 53.0	268.8 179.5 106.6 49.8	277.6 190.0 115.2 59.8
8	L^1L^2 L^2L^3 L^3L^4 L^4L^5 L^5L^6	253.9 182.7 121.7 70.0 33.1	255.9 186.4 125.1 77.0 41.4	285.8 204.9 136.9 80.3 36.9	290.3 212.2 143.4 87.1 46.4	324.0 232.7 155.1 92.0 42.2	328.4 251.8 166.2 100.8 52.3
9	L^1L^2 L^2L^3 L^3L^4 L^4L^5 L^5L^6	296.0 224.0 160.9 106.7 61.8	298.0 227.5 165.6 111.1 70.6	333.3 251.4 180.7 120.4 70.1	333.5 252.0 188.7 127.3 77.4	378.6 285.8 204.6 136.8 78.6	379.0 286.5 212.8 147.3 89.6

It will be noted that the equivalent uniform load method gives shears closely approximating those for the engine diagram for the main diagonals and exhibits a slight excess for counters which if made of stiff members would not alter the sections at all. Hence the method is practical, gives values on the side of safety and is a time saving device: The computation has been carried out for trusses of from four to nine panels. The exact shear by the engine diagram being noted in one column and the corresponding shear as figured by the equivalent uniform load in the other column.

5. Impact: Impact may be defined as an allowance to cover the suddenness of the application of the load and the dynamic effect of unbalanced driving wheels, the lateral effect of the rotating tendency of the locomotive about a vertical axis and also a horizontal axis under the alternating thrust and pull of the reciprocating engine.

The mathematical determination of the rotative couple has been treated at some length by Weisbach yet its precise translation into elastic strain is too complex for other than approximate determination.

As a railroad train passes over a bridge structure, a certain element of time is required to overcome the inertia of the mass in producing the greatest deflections of the parts and of the structure as a whole. The greatest elongation under tension, or deformation under compression, is a thermo-dynamic function such that the mechanical energy represented by the transfer of the kinetic heat energy of molecular vibration in the resisting metal to the potential energy of deformation of strain is double the external mechanical work of the load when the stress is tension, and the reverse when it is compression. This excess energy is radiated to the surrounding medium or the absorption of heat energy from it takes place. Accordingly, there is an instantaneous or kinetic deformation and a static deformation which involves the after thermal effect.

The relative amount of energy involved, as shown by Kelvin's thermo dynamic theory, is approximately double that of the work of the external applied forces. The full development of the static deformation requires an element of time, which in connection with overcoming the inertia of the mass, may be approximately estimated at 1 second.

If the train is traveling nearly 60 miles an hour, its speed may be taken at 100 feet per second, and the vertical impact on a member would approximate

$$L\left(\frac{100}{A+100}\right)$$

in which

*L = live load stress; (statically determined).

A = length of track loaded, for maximum effect, when the initial dead load stress equals zero.

However, if the member is under dead load stress, the approximate vertical impact strain would equal

$$L\left(\frac{{}^{*}L}{D+L}\times\frac{100}{A+100}\right)$$

The rolling load tends to set up structural vibration laterally as well as vertically, and the total impact is the resultant of the vertical and horizontal effects combined. In the ordinary pin-connected bridge, with a train speed of about 60 miles or more per hour, the lateral impact may be assumed to be approximately equal to the vertical impact effect. Therefore, the formula for total impact is

$$I=L\left(\frac{L}{D+L}\times\frac{140}{A+100}\right)$$

The vertical effect of impact would vary approximately as the speed of the train, whereas, the lateral effect would vary roughly as the square of that speed. Accordingly, if the rational impact allowance of the high-speed passenger train is to be compared to that of the ordinary freight train, which travels about half as fast and the lateral impact of which would be one-fourth as great, these formulas would immediately reduce to

$$I=L\left(\frac{L}{D+L}\times\frac{55}{A+50}\right)$$

It therefore appears that the high speed passenger train impact for a Cooper E 60 load would equal an E 75 loading for freight train speed. As the excessively heavy Malley engines are not used for fast passenger service, there is little call for change in the present typical engine loading when coupled with a high speed impact formula. The Cooper generalized standard furnishes a uniform method of rating the capacities of bridges which is of no small value. In fact, it requires but slight rectification of the irregularity of its curve of uniform loading to render it sufficient for all practical requirements.

IMPACT FORMULAS

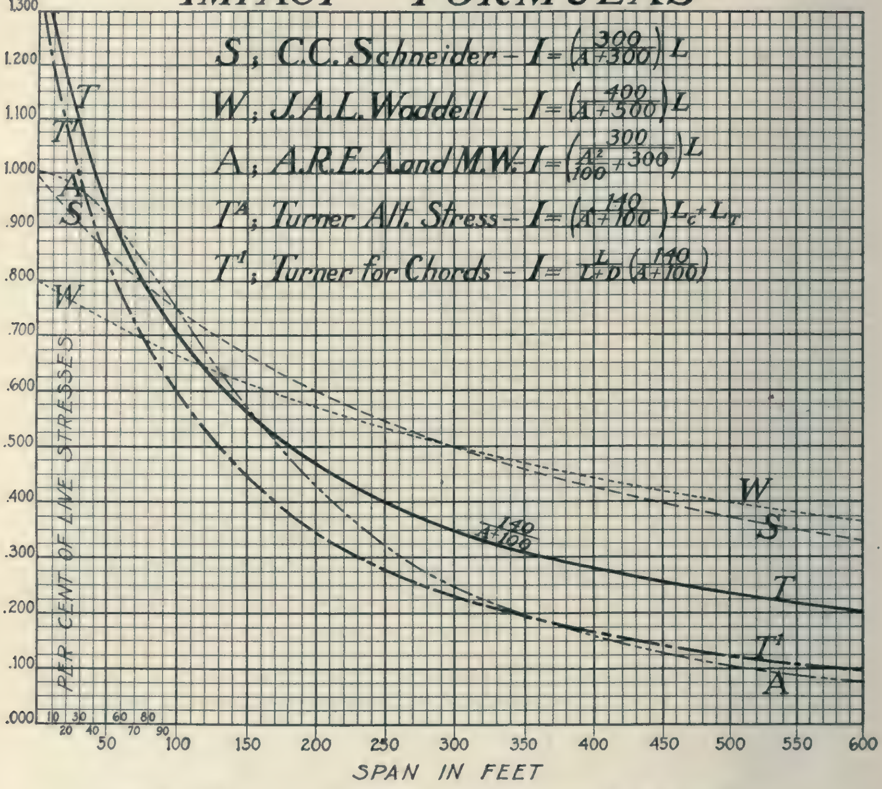
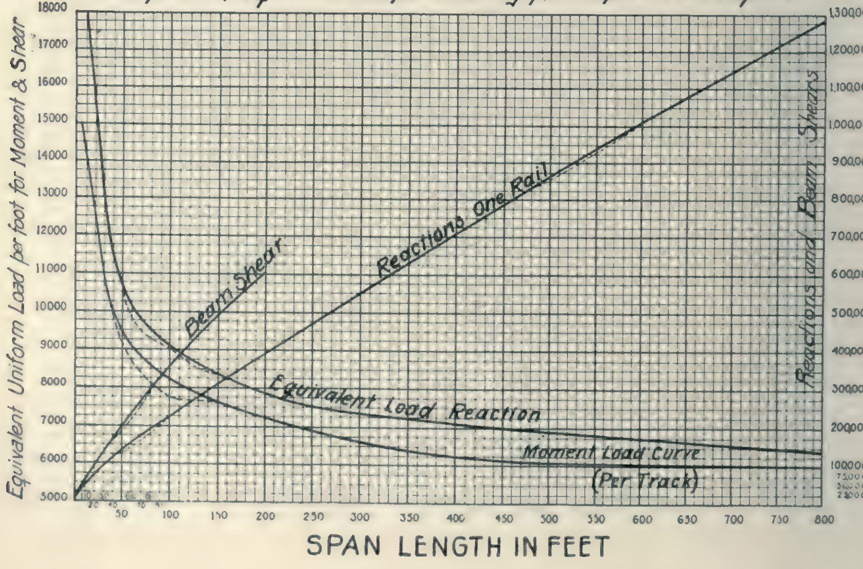


Diagram of Equivalent Uniform Loading for Cooper's E 60 Engines.



For range of stress from tension to compression, or vice versa, the first member of the formula becomes unity and may be disregarded and the live load should be taken in determining the impact as the sum of the tension and compression, or if L_c = the compression and L_t = the maximum tension numerically, then

$$I=\left(\frac{140}{A+100}\right)\times(L_c+L_t)$$

In the accompanying chart of impact formula the dot dash curve is a convenient approximation for chord members.

The Schneider, Waddell and the later A. R. E. A. formulas are plotted in dash-dash lines. Each of the latter give insufficient values for short stringers where the dynamic effect is greater than 100 percent. They fail to differentiate between the cases of negligible dead load on the one hand and a large percentage on the other. In the former case only the section of metal provided for the live stress resists impact strain while in the latter the additional section provided for dead load stress functions pro rata in its limitation. They are further deficient in provision for the increased impact when the range of stress is from tension to compression. A consistent curve of variation results from the correction of these deficiencies in strong contrast to the *OG* curve with the present A. R. E. A. specification.

For double track bridges these impact values remain the same for the floor system but should be reduced to 70 percent of those for the single track bridge, on the following grounds:

First, that it will rarely occur that both tracks are loaded in such wise as to produce the maximum figured stresses. Second, even tho they are so loaded, the impact vibration of the two train loads would interfere rather than synchronize in such wise as to produce a maximum and for these reasons an allowance of 70 percent of the impact value is on the safe side.

6. The Dead Weight of the Structure Varies with the depth of the floor system, economic depth of the trusses, economic arrangement of the panels, and the type of structure.

Evaluation of these variables will be undertaken in the later chapters but for the present article the dead weight of structures built in accordance with common practice is approximated by the following formulas for E 60 loading.

For deck girders the weight of steel per lineal foot (w) in lbs. may be computed in terms of L, the length of the span in feet:

For the Deck Girder $w = 15L$ in lbs. per lineal foot of span.
For half thru Girders $w = 15L + 600$.
For Truss Spans $w = 11L + 750$.

Trautwine gives the following diagram for the weights of steel railroad bridges based on Northern Pacific design by Mr. Modjeski (from the Journal, Western Society Engineers, Feb., 1901).

The loading consists of two locomotives of 146 tons each uniform train load of 4,000 pounds per foot of track. The weights comprise the girders or trusses of one single track span with bracing, floor system and end bearings. For the total weight for computation add 500 pounds per lineal foot for the wood floor system. For the pin connected spans 130 to 200 feet, three dash-dash diagrams show respectively the weight of two trusses only, two trusses and lateral bracing, and two trusses, bracing and metal floor. The solid curve includes the weight of end bearings in addition to the foregoing.

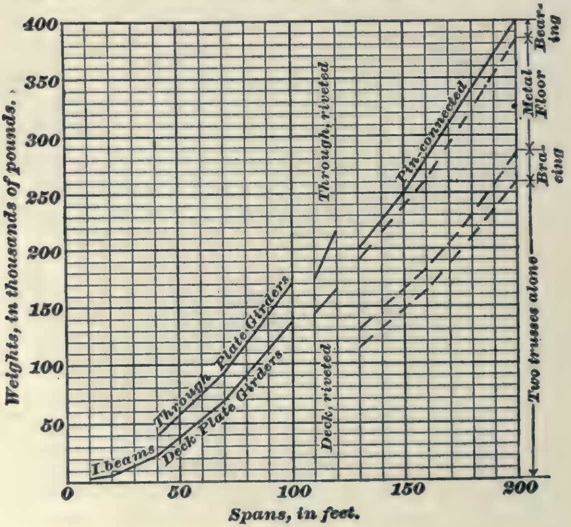
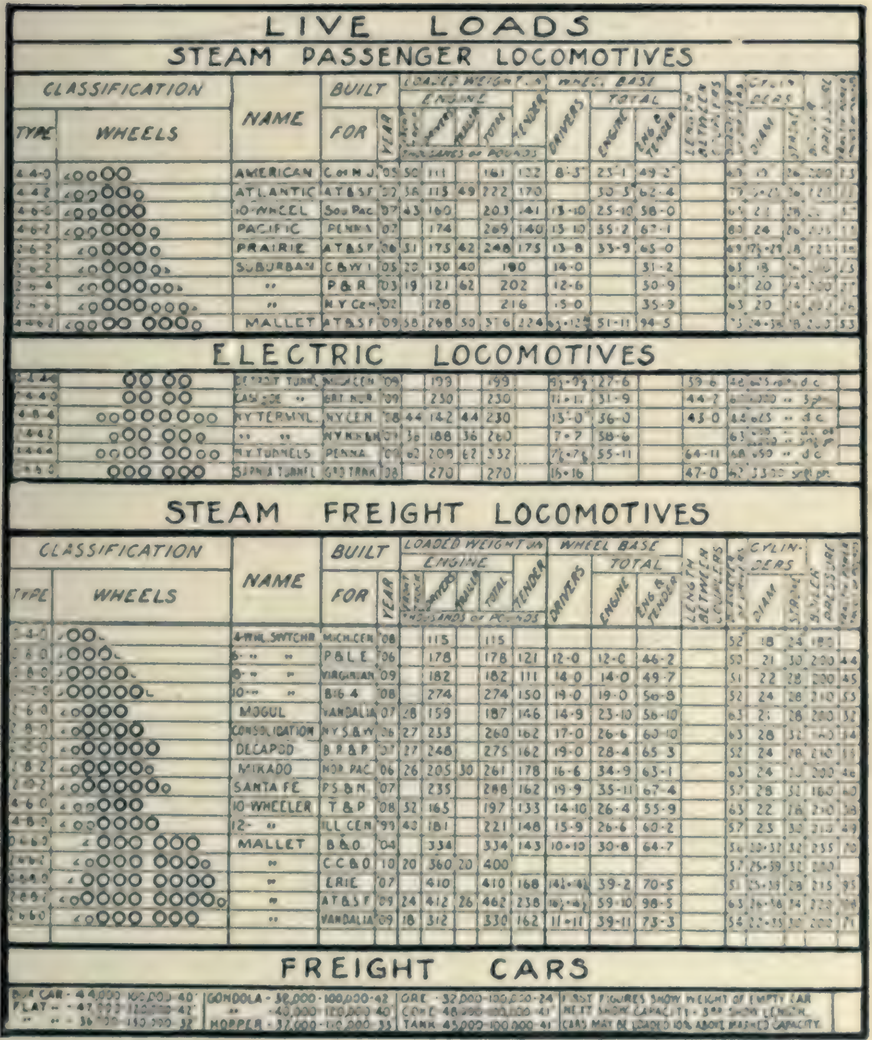


Fig. 66.

7. Classes and Weights of Locomotives and Freight Cars in use are tabulated in the accompanying diagram for both freight and passenger service.

It is to be noted that the freight car load in general materially underruns 6,000 pounds per linear foot. Only the flat car fully loaded is equivalent to 6,000 lbs. per foot. In isolated cases of overload it may for a single car length overrun 6,000 lbs. by eight percent. Ore cars may run between five and six thousand pounds per linear foot. With the electric locomotive the impact is about two-thirds that for the steam locomotive.



LIVE LOADS

STEAM PASSENGER LOCOMOTIVES

CLASSIFICATION		NAME	BUILT		LOADED WEIGHT IN THOUSANDS OF POUNDS: ENGINE					WHEEL BASE			LENGTH BETWEEN COUPLERS	[illegible]	CYLINDERS		[illegible]	[illegible]
TYPE	WHEELS		FOR	YEAR	[illegible]	DRIVERS	TRAILER	TOTAL	TENDER	DRIVERS	TOTAL: ENGINE	TOTAL: ENG & TENDER			DIAM	STROKE		
4-4-0	<ooOO	AMERICAN	C of N J	05	50	111		161	122	8-5	23-1	49-2		[illegible]	[illegible]	26	200	[illegible]
4-4-2	<ooOOo	ATLANTIC	AT&SF	[illegible]	36	115	49	222	170		30-5	62-4		79	[illegible]	26	[illegible]	[illegible]
4-6-0	<ooOOO	10-WHEEL	Sou Pac	07	43	160		203	141	13-10	25-10	58-0		[illegible]	21	28	[illegible]	[illegible]
4-6-2	<ooOOOo	PACIFIC	PENNA	07		174		269	140	13-10	35-2	67-1		80	24	26	[illegible]	[illegible]
2-6-2	<oOOOo	PRAIRIE	AT&SF	06	31	175	42	248	175	13-8	33-9	65-0		69	[illegible]	28	[illegible]	[illegible]
2-6-2	<oOOOo>	SUBURBAN	C&WI	05	20	130	40	190		14-0		31-2		63	18	[illegible]	[illegible]	[illegible]
2-6-4	<oOOOoo>	"	P&R	03	19	121	62	202		12-6		30-9		61	20	24	[illegible]	[illegible]
2-6-6	<oOOOooo>	"	N Y Cen	02		128		216		15-0		35-3		63	20	[illegible]	[illegible]	[illegible]
4-6-6-2	<ooOO OOOo	MALLET	AT&SF	09	38	268	50	376	224	[illegible]	51-11	94-5		73	24-38	28	[illegible]	53

ELECTRIC LOCOMOTIVES

TYPE	WHEELS	NAME	FOR	YEAR	[illegible]	DRIVERS	TRAILER	TOTAL	TENDER	DRIVERS	ENGINE	ENG & TENDER	LENGTH BETWEEN COUPLERS	[illegible]
[illegible]	OO OO	DETROIT TUNL	MICH CEN	09		193		199		[illegible]	27-6		39-6	[illegible] d c
[illegible]	OO OO	CASCADE "	GRT NOR	09		230		230		[illegible]	31-9		44-2	[illegible]
4-8-4	ooOOOOoo	NY TERMNL	NY CEN	08	44	142	44	230		13-0	36-0		43-0	[illegible] d c
[illegible]	oOO OOo	" "	NYNH&H	[illegible]	36	188	36	260		7-7	38-6			63 [illegible]
[illegible]	ooOO OOoo	NY TUNNELS	PENNA	[illegible]	62	205	62	332		[illegible]	55-11		64-11	68 [illegible] d c
[illegible]	OOO OOO	[illegible] TUNNEL	GRD TRNK	08		270		270		16-16			47-0	[illegible] 3300 [illegible]

STEAM FREIGHT LOCOMOTIVES

CLASSIFICATION		NAME	BUILT		LOADED WEIGHT IN THOUSANDS OF POUNDS: ENGINE					WHEEL BASE			LENGTH BETWEEN COUPLERS	[illegible]	CYLINDERS		BOILER PRESSURE	[illegible]
TYPE	WHEELS		FOR	YEAR	FRONT TRUCK	DRIVERS	TRAILER	TOTAL	TENDER	DRIVERS	TOTAL: ENGINE	TOTAL: ENG & TENDER			DIAM	STROKE		
0-4-0	oOO	4-WHL SWTCHR	MICH CEN	08		115		115						52	18	24	[illegible]	
0-6-0	oOOO	6- " "	P&LE	06		178		178	121	12-0	12-0	46-2		50	21	30	200	44
0-8-0	oOOOO	8- " "	VIRGINIAN	09		182		182	111	14-0	14-0	49-7		51	22	28	200	45
[illegible]	oOOOOO	10- " "	BIG 4	08		274		274	150	19-0	19-0	50-8		52	24	28	210	55
2-6-0	<oOOO	MOGUL	VANDALIA	07	28	159		187	146	14-9	23-10	56-10		63	[illegible]	28	200	[illegible]
2-8-0	<oOOOO	CONSOLIDATION	NYS&W	06	27	233		260	162	17-0	26-6	60-10		63	28	32	[illegible]	[illegible]
[illegible]	<oOOOOO	DECAPOD	BR&P	[illegible]	27	248		275	162	19-0	28-4	65-3		52	24	28	[illegible]	[illegible]
2-8-2	<oOOOOo	MIKADO	NOR PAC	06	26	205	30	261	178	16-6	34-9	63-1		63	24	[illegible]	200	[illegible]
2-10-2	<oOOOOOo	SANTA FE	PS&N	07		235		288	162	19-9	35-11	67-4		57	28	32	[illegible]	[illegible]
4-6-0	<ooOOO	10-WHEELER	T&P	08	32	165		197	133	14-10	26-4	55-9		63	22	26	200	[illegible]
4-8-0	<ooOOOO	12- "	ILL CEN	99	40	181		221	148	15-9	26-6	60-2		57	23	30	[illegible]	[illegible]
0-6-6-0	<OOO OOO	MALLET	B&O	04		334		334	143	10-10	30-8	64-7		56	20-32	32	235	[illegible]
2-6-6-2	<oOOO OOOo	"	CC&O	10	20	360	20	400						57	25-39	30	[illegible]	
0-8-8-0	<OOOO OOOO	"	ERIE	07		410		410	168	[illegible]	39-2	70-5		51	25-39	28	215	95
2-8-8-2	<oOOOO OOOOo	"	AT&SF	09	24	412	26	462	238	[illegible]	59-10	98-5		63	26-38	34	[illegible]	[illegible]
2-6-6-0	<oOOO OOO	"	VANDALIA	09	18	312		330	162	11-11	39-11	73-3		56	22-35	[illegible]	200	[illegible]

FREIGHT CARS

BOX CAR - 44,000 - 100,000 - 40'	GONDOLA - 38,000 - 100,000 - 42	ORE - 32,000 - 100,000 - 24	FIRST FIGURES SHOW WEIGHT OF EMPTY CAR
FLAT " - 47,000 - 120,000 - 42'	" - 40,000 - 100,000 - 40'	COKE - 48,000 - 100,000 - 41'	NEXT SHOW CAPACITY - 3RD SHOW LENGTH
" " - 36,000 - 150,000 - 32'	HOPPER - 37,000 - 110,000 - 35	TANK - 45,000 - 100,000 - 41	CARS MAY BE LOADED 10% ABOVE MARKED CAPACITY.

8. Lateral Forces to be taken into consideration in bridge design are wind and the lateral component of the impact from the rolling load.

An investigation extending over a number of years from 1883 to 1890 was made by Sir Benjamin Baker and his assistants of wind pressures recorded during violent gales at Inchgarvie. Those registered by the large fixed gage 15'x20' in area ranged from 12 to 35 pounds per square foot. On one occasion the barometer fell over

¾ of an inch within an hour. The pressures measured by the revolving gage and a small fixed gage were higher than those registered on the large gage and the error of the gage reading for the smaller gages was investigated and the arrangement of the recording apparatus altered since it was found that with the lever multiplication pointer the high readings were in a large part due to the fact that the lever acquired some momentum from a sudden gust, over shooting the mark and indicating a fictitious pressure. After the horizontal lever was done away with the highest recorded pressure with the small gages was 35, 41 and 27 pounds per square foot respectively.

The gale of March 31, 1886, gave the following results: Upon a small fixed gage 31 pounds to the square foot; upon a revolving gage 26 pounds; center of the large board 28½ pounds; board corner 22 pounds. Average over the board 19 pounds. From this it appears that the high wind pressures come in gusts differing from a state of even pressure extending over large areas.

After the central towers of the bridge had been carried up the full height, revolving gages, one at the Northeast and one at the Southwest corner of each tower, were put up and it was found that the pressures recorded varied as much as 10 and 12 pounds between the different piers, sometimes one and sometimes the other showing the higher registration.

Horizontal wind pressure in a gale or a hurricane may accordingly be conservatively taken at 30 pounds per square foot over the exposed area of one truss and floor in elevation and one and one-half times this area for two trusses for spans up to 200 feet and two-thirds of this amount for a 400-foot span and over. Intermediate values for intermediate spans.

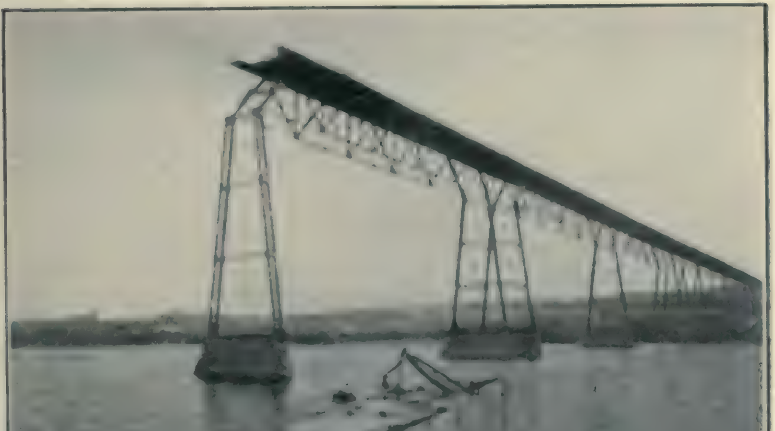
General Greeley, of the United States Signal Service, in 1890 after the Louisville, Ky., tornado, stated:

> "As bearing upon the strength of structures necessary to withstand tornadic winds, it is important to note that there have been very few cases recorded of wind velocities in the United States where the pressure of the wind, according to the latest investigations and accepted formulae, exceeded 16 lbs. to the square foot."

The severest effects of wind occur in the paths of tornadoes, these, however, are very limited in their width, the high pressures or destructive effects being limited to a few hundred feet. Being a rotating force it exerts its lateral pressure in a manner differing from that of the gale and depends in a large measure upon the suction of the vortex and the rapidity of its motion.

C. Shaler Smith reported a tornado at the St. Charles bridge which exerted a pressure of 52 pounds on small objects and 84 pounds in the vicinity which did not injure the bridge altho its bracing was proportioned for only 30 pounds per square foot of exposed surface. He also found in following up the paths of several tornadoes but one case where 60 feet in width was insufficient to cover the path within which the computed pressure exceeded 30 pounds.

The accompanying figure represents the destruction of a Section of the Smith Avenue Viaduct, known as the High Bridge in St. Paul, looking Northwest. The author figured this wreck was caused by a pressure of approximately 30 pounds acting upward at an angle of about 45°. Lifting the end of the 250-foot span from the top of the tower at the expansion shoe where it was not bolted down, the wind by its leverage twisted the fixed tower from its base, resulting in the wreck shown in the figure. Such upward inclination of the wind force should be given consideration in design as it gives rise to the most destructive effects.



The nature of the pressure in cyclonic storms is better understood by considering its effects upon buildings. As the vortex passes over a frame building the atmospheric pressure on the outside is suddenly removed by the suction of the vortex and the inside pressure causes destruction similar to that of an explosion. Windows are blown outward and a complete wreck commonly results. Such examples as the removal of the leeward roof surface of a barn leaving the windward roof and rafters standing is not uncommon.

The extreme magnitude of the pressures thus developed in a building probably never exceeds 75 pounds per square foot and for that reason the damage to the reinforced concrete structure in a tornado is confined to the destruction of windows and partitions rather than serious damage to the building as a whole.

Mr. Theodore Cooper in discussing the writer's analysis of the wreck of the High Bridge at St. Paul, Transactions Am. Soc. C. E., December, 1904, would provide for an excess pressure of 50 pounds per square foot over a width of 150 feet and a 15-pound pressure for the remainder of the span for long span structures 1,200 to 2,000 feet in length.

Lateral forces of impact have increased with the increase in engine loading and the increased speed of the present day train over the highest speed customary with trains of thirty and forty years ago. The specification more commonly used in the 90s was that of Theodore Cooper in which we find the following requirement:

> "To provide for wind strain vibration, the top lateral bracing in deck bridges and the bottom lateral bracing in through bridges shall be proportioned to resist a lateral force of 450 pounds for each foot of span, 300 pounds of this to be treated as a moving load. The bottom lateral bracing in deck bridges and the top lateral bracing in through bridges shall be proportioned to resist 150 pounds for each foot of span."

According to Dubois this was arrived at as follows:

> "Exposed area of the train is about 10 square feet for every foot in length; at 30 pounds per square foot this gives 300 lbs. for every foot of length which shall be treated as moving load. The truss would probably not have more than 10 square feet of exposed surface per lineal foot and taking half of this on each chord, we have a fixed load of 150 pounds per lineal foot for the top and bottom chord, or 150 pounds fixed load per foot for the unloaded chord, 450 pounds per lineal foot for the loaded chord of which 300 is live and 150 fixed."

But in providing merely for the wind load no allowance is included for the lateral impact of the heavy locomotive at high speed nor is there any allowance for the column action of the top chord compressions resisted by the lateral bracing.

9. Lateral Force for a Rigid Design for spans up to 600 feet may be specified as a uniform wind load of 200 pounds per lineal foot of span on both the top and bottom chord of the truss bridge up to 200 feet in span and 25 pounds per foot additional load for each 50 feet increase in span above this limit. In the floor of the thru bridge or deck bridge an additional shear on the laterals equal to the sum of that due to the uniform wind load plus one-tenth of the equivalent uniform load for a single track fully loaded. For the top chord laterals of the thru bridge a shear in addition to that for the uniform wind load equal to three percent of the compressive stress in the end chord member, for all panels to provide for the column action of the chords as latticed by this bracing.

For the deck plate girder of 80 feet in span this would give a shear on the end lateral of approximately 50,000 pounds. The section of the bracing customarily provided by the experienced designer meets this requirement altho it is not required by the specifications in common use. The scientific rule should call for such sections as general experience warrants and judgment of the most experienced designers provide.

Since the impact allowance on the chords takes into consideration both the vertical and lateral impact of the loading it is necessary to compute only the additional wind load that is brought upon the loaded chord by the wind pressure on the train itself. The exposed area of the train may be taken as 10 square feet per lineal foot and the wind pressure at 30 pounds per square foot for spans up to 200 feet in length, reducing to 20 pounds for spans as long as 600 feet or over and for intermediate span lengths, intermediate values.

As excessive wind loads are rare and momentary in action an increase in working stress for wind strains on the chord of twenty-five percent may be allowed in computing the total area from the combined stresses of live and dead loads, impact and wind strains.

10. For Bridges on Curves provision must be made for the increased load carried by any truss, beam or stringer due to the curvature of the track and the eccentricity of the application of the load caused thereby. In addition to this eccentricity the centrifugal force of the train traveling at a speed commensurate with the curvature should be allowed for as provided in the A. R. E. A. specification:

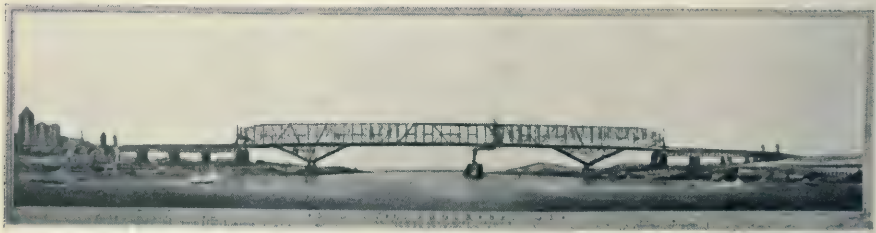
Degree of curve......	0° 21′	0° 40′	1°	2°	3°	4°	5°	6°	7°	8°	9°	10°	11°	12°
Percentage	2½	5	7½	10	10	10	10	10	10	10	10	10	10	10
Speed in miles per hour	80	80	80	65	53	46	41	38	35	33	31	29	28	27

11. The Subject of the Aesthetic or Science of the Beautiful in Bridge Design is a matter regarding which there is such a diversity of opinion that we can well appreciate the homely truth of the old woman's statement "there is no accounting for taste" when she kissed the cow. By coupling the science of the beautiful with economic considerations definiteness is imparted to this otherwise abstruse or occult subject matter.

It may be asserted as a cardinal principle or fundamental law that the lines of greatest economic strength are essentially graceful and beautiful. The converse of this rule that a graceful design is essentially economical does not hold true, because economy requires a type of superstructure in keeping with foundation conditions and rigid adherence to certain fixed proportions or ratios of depth to span length combined with economic triangulation of the truss members which may vary thru a wide range without material detriment to aesthetic appearance.

Because aesthetic appearance proper permits this elastic variation, while maximum economy does not, it is possible for an ugly design to be more economical than one merely aesthetic when it approaches more closely to those proportions determinant of economy. We may thus condemn the ugly design as lacking in maximum economy from its appearance but we may not assume that an aesthetic design is most economical, without first investigating its approach to economic proportions.

So little are these fundamentals understood that many bridge engineers look for the cheap design in the quintessence of ugliness. This may be illustrated from a comparison of two competitive cantilever designs submitted for the Sydney bridge. As both propose the use of the same heat treated eye bars with very high working stresses, a direct comparison of weight is to the point:



Est. Weight, 49,115 tons.

Est. Weight, 38,064 tons.

A mathematical method for the verification of the fundamental law that the lines of strength are essentially graceful when forming a truss triangulation of maximum economy will be developed in a following chapter from the variation of the summation of strain areas. It is considered by all that the design should be as symmetrical as the nature of the location permits and of a type in harmony with the surroundings.

12. Summary: It would seem in reviewing the literature of bridge engineering that the writers on this branch of science have endeavored to make the subject as complex and as involved from the mathematical standpoint as possible leading to a performance one expects from the youngster just out of school in accurately figuring the heavy cut and fill on the railroad right of way to the ten-thousandth part of a yard per station when the measurements made with the prismatic formula would not warrant an expectation of its accuracy within three or four yards per station.

Thus the chronic constipation of ideas regarding wheel loads of the bridge fraternity has been relieved only by a continuous diarrhoea of computation, expensive in time and wasteful of energy which should be more fittingly employed in the devise of better details, simpler shop work, and fewer joints conducive to greater rigidity.

The computation of stresses by exact methods to the nearest kip affecting the section but a fraction of a percent when the rolling mill does not guarantee the sections called for within a two percent tolerance is typical of this inconsistency.

The elimination of this drudgery by a simple equivalent load diagram which departs from the generalized engine loading an amount sufficient to cover a material variation in the driving wheel spacing is presented in the foregoing discussion and a rational type of impact formula has been developed which recognizes the limitation of the vibratory impact effect by the existing dead stress in the member. It further takes into consideration the range of stress in the web member where it changes sign from tension to compression by an increase in the impact allowance in proportion to the entire range of stress thus fittingly providing for an impact strain greater than any formula heretofore proposed for such members while reducing the excessive impact allowance for chords provided by the Waddell formula and that of Mr. Schneider for the long spans. It moreover increases the impact for short stringers above these formulas in keeping with the suddeness of application of the load and the shock incident thereto.

The writer's observation of eye-bar counter ties stretched any where from one half to one inch in the maintenance of old overloaded bridges has taught him to appreciate the desirability of liberal sections for such members and the elimination of flim flam sections for counter strains as C. C. Schneider used to classify them.

In dealing with lateral stresses the wind effect on the long span has been specified in keeping with the studies and observations of Sir Benjamin Baker and the writer's study of the effects of cyclonic storms. The suggested specification does not over-load the chord while it does require ample section for stiffness of the lateral bracing.

CHAPTER XXIV

ECONOMIC THEORY OF DESIGN OF BEAMS AND PLATE GIRDERS

1. The theory of beams of uniform cross section simply supported, restrained and continuous with the stress line analysis of those of constant section has been developed in Chapter VIII, Sec. II. The modification of the theory in case of variable moment of inertia has there been treated in its broader aspects leaving for present consideration questions of economic depth, web thickness, stiffening, connections and sway bracing in the practical bridge structure and for convenience in computation and preliminary reports the development of simple dead weight formulas or charts for more convenient reference.

2. The Economic Ratios of Web to Flange Area may be investigated from the formula of Chapter VIII, Sec. II, for strength of rolled beams under 16,000 lbs. fiber stress.

$$\text{Safe load } W = \frac{K\,w\,d''}{L}$$

In which W is the safe load in thousand pound units. K a coefficient depending on the ratio of the weight of the web to that of the flanges, w equaling the weight of metal per lineal foot, d'' the depth in inches and L the span in feet.

Now K equals unity for the minimum section, it decreases as the web is thickened up in the maximum section and increases in the Bethlehem girder beam where a large percentage of the total weight is concentrated in the flange area. Thus:

For Bethlehem 12″ 55 lb. Girder Beam
Web is .37″ thick $C' = 768{,}000 \quad I = 432$

For Carnegie 12″ 55 lb. Standard Beam
Web is .81″ thick $C'' = 570{,}000 \quad I = 321$

For Carnegie 18″ 55 lb. Standard Beam
Web is .46″ thick $C''' = 943{,}000 \quad I = 795$

Thus by cutting the thickness of the web in half, and by widening out the flange from 5.6″ to 9.75″, the bending strength of the Carnegie Standard 12″ beam has been increased thirty-four percent in the Bethlehem Section, as is evident from the ratio of the coefficients of strength C', C''.

The area of web equals the area of flange in the 12″-55 lb. Standard beam but in the Bethlehem girder beam the area of web is one-third the area of flange.

We will now compare these two beams with the Standard 18″-55 lb. in which the area of flange substantially equals the area of the web:

The coefficient of strength of the 18″ beam is one and two-thirds as great as that of the Standard 12″-55 lb. beam and one and a third times as great as the 12″-55 lb. girder beam.

From this comparison it is clear that the usual computation of the ratio of the area of web to that of flange is broadly not a determinant of economic strength but becomes such a determinant when the web section is THIN enough to secure the greatest depth consistent with safety against buckling or crippling.

3. The Economic Depth of a Girder with a Fixed Thickness *of Web*, requires that *the weight of the web equals the weight of the flanges*. The section of the flanges for a fixed fiber stress increases directly as the depth decreases and if the thickness of the web be fixed its weight reduces as the depth decreases, and vice versa.

Let D equal the economic depth and W equal the minimum weight half web and half flange. Then if the depth be reduced half—

$$\begin{array}{lcl} \text{Flange Weight} & = & W/2 \times 2 = W \\ \text{Web Weight} & = & W/2 \div 2 = W/4 \\ \hline \text{Total Weight} & = & 5/4W \end{array}$$

or an increase of one fourth for a reduction of economic depth by half.

Now if D be reduced a quarter then

$$\begin{array}{lcl} \text{Flange Weight} & = & W/2 \times 4/3 = 2/3W \\ \text{Web Weight} & = & W/2 \times 3/4 = 3/8W \\ \hline \text{Total Weight} & = & 25/24W \end{array}$$

If D be increased one-fourth, then

$$\begin{array}{lcl} \text{Flange Weight} & = & W/2 \times 4/5 = 2/5W \\ \text{Web Weight} & = & W/2 \times 5/4 = 5/8W \\ \hline \text{Total Weight} & = & 41/40W \end{array}$$

If the departure from D be $+ 10\%$ then the increased weight in like manner is $W/220$ and if it be decreased 10% then the increase in weight is $W/180$. Therefore under the preceding premise a variation of plus or minus 10% from equal weight of the flange and web may make but half a percent difference in total weight.

It should be noted, however, that increase in depth over that which is most economical will not involve *increase in web thickness* while a decrease may *necessitate* such increase, and moreover in the shallow beam f may have to be decreased to secure desirable rigidity under live load, i. e., increased flange area may be necessary over that required for quiescent loads.

4. Stiffness of a simply supported rolled or built beam varies according to the formula derived in Chapter VIII, Sec. 11.

$$\triangle = KfL^2 / Ed\,(1 + g)$$

in which K is a coefficient depending upon the manner of loading, f the maximum fiber stress, L the span, d the depth, E Young's modulus and g the proportion of deflection caused by shear.

K may be taken as substantially 1/5 for locomotive loading since its exact value varies from 8/48 for concentrated central loading to 10/48 for uniform load and 11/48 for equal concentrated loads at the quarter points.

For a Bethlehem girder 30″ × 175 lbs., web 67″ thick as computed, pages 20-24, Sec. II, $g = 0.24$ for a 15-foot span, 0.13 for a 20-foot span, 0.06 for a 30-foot span. Now as g depends upon the ratio of transverse flange area resisting moment to the half longitudinal web area resisting horizontal shear with a built girder having a three-eighths web and the same flange area these proportions of shear deflection would be doubled up. However, the ratio of flange area to web area with this Bethlehem girder is two to one a wide departure from economic equality, hence for economic area ratios in simple span girders with $L/d = 1/6$, $g = .12$ app. and for $L/d = 1/8$, $g = .06$.

Therefore, with economic built girder designs the deflection caused by shear distortion may be disregarded as small but for an uneconomic too shallow depth, shear distortion adds largely to the deflection unless a liberal provision has been made by increase of *flange area over that required for quiescent load to reduce the deflection and vibration under rolling loads.

Moment deflections in terms of span length are simply computed from the foregoing formula: Thus, if $f = 16{,}000$ lbs. per square inch and $d = L/8$ by substitution

$$D\text{, the moment deflection} = \frac{16{,}000\ L^2}{5\ E\ L/8} = \frac{L}{1200}\ \text{approx.}$$

---

*If the web be not overstrained the deflection is more economically reduced by adding to the flange area, than to web section.

or for the 30″ Bethlehem girder beam 20-foot span, it would be two-tenths of an inch or with a 48 × 3/8 web (a mean approximation to economic depth) the deflection reducing proportionally to the increase in *d* would be but twelve hundreths of an inch.

This represents a most radical difference in deportment requiring no computation for its appreciation if the observer stations himself consecutively upon spans of these relative depths during passage of a fast train.

5. A Series of Continuous Rolled Beams or Plate Girder Spans require less metal and are much stiffer than a similar series of simple spans. The graphical derivation of the law of moment magnitudes, page 58, Sec. II, discloses this fact at a glance.

By continuity the stringer attains its maximum rigidity so that by its relative freedom from vibration the completed structure gives the impression of being the stiffest bridge in its neighborhood even though it may not have been designed to carry the heaviest loading.

This increase of stiffness with varying load from two to five fold over non-continuous stringers permits a shallower floor construction where such is advantageous.

The deflection in an interior span under uniform loading due to bending moment is one-fifth as great as for a simply supported span of the same uniform cross section and but one-fourth as great under a concentrated central load.

Interference of shear detrusion between the cantilever and suspended span portion of the continuous girder reduces the moment deflection an amount dependent upon the ratio of L/d as investigated in Arts. 11 and 12, pages 69-70, Sec. II.

The balance of tension and compression of the fiber stress areas are modified by this interference of detrusion so that for an I section under a central load when $L/d = 10$, the tensile stress at the top of the restrained beam over the support is but eight-tenths the compression at the bottom and at mid span a like reduction in the tension occurs.

Under a uniform load continuity reduces the moment at the center of the intermediate span to a third of the moment when simply supported and by half for equal concentrated loads at the center of each span. With the live load advancing the adjacent forward span unloaded is subject to negative bending reducing the restraint and efficiency of continuity over the support. Again, with the engine loading consecutive spans are subject to loads of varying intensity.

Equally important in considering the economy of continuity is the practical question of the workmanship or initial rigidity of the milled joint between the stringer end and the floor beam. A slight yielding in coming to a bearing renders a flange section of seventy percent as great as required for a simply supported beam sufficient over the support and excessive at mid span.

If the weight of the flanges equals that of the web, the total saving out of 30% reduction in flange area approximates 12% of the weight of the stringer since the thirty percent of flange represents but fifteen percent of the weight of flange and web combined to be in turn reduced by the weight of the splice of the tension flange over the beam.

Advantage cannot be taken of the interference of detrusion to reduce the tension flange in the stringer because the section is ordinarily made of two angles having constant or uniform section which is called on to resist a maximum compression at the support when subject to a maximum tension at mid span and vice versa.

6. For the Bridge Stringer Depth Ratios of L/D for E60 loading satisfactory from the standpoint of rigidity should not under run one-seventh of the length for non-continuous stringers nor one-eleventh in case they are made fully continuous by splicing the flanges over or thru the supporting beams. For light narrow gage railroad, these ratios may be reduced to one-eighth for discontinuous and one-twelfth for continuous stringers. Any reduction in this depth ratio should be accompanied by a pro rata reduction in the allowed fiber stress to maintain the standard degree of stiffness and render the same impact allowance applicable.

Floor Beams when rigidly riveted to the truss frame by bending cause a torsional moment upon the truss and hence should be stiffer than the stringer.

A limitation of the depth to $L/4$ for an E60 loading and $L/5$ for a narrow gage E25 loading will give satisfactory results with a decrease in flange stress in proportion to any decrease in relative depth.

For Deck Plate Girders of 90 to 100 feet in span one-tenth to one-eleventh the span length is satisfactory from the standpoint of stiffness while for spans intermediate between 40 feet and one hundred feet proportionate decrease in depth between one-seventh and one-tenth the span are good ratios for E60 loading.

7. The State of Stress in the Plate Girder Web differs from that in the rolled beam analyzed in Sec. II, pages 79 and 80, when the relatively thin web is stiffened against buckling. Without stiffners it is agreed that the principal stresses of tension and compression follow substantially the same lines as in a homogeneous eye beam. Concentration of metal in the flanges affects the shear stress areas and flattens the curves of greatest horizontal and vertical shear.

The displacement and equi-potential curves have some resemblance to the erroneous representation of the principal web stresses presented by Johnson* and some others as tho extending across the web at 45° like the slats of a TOWNE lattice truss. That knicking a uniform section modifies the intensity of the principal stresses at the locus of abrupt change has long been recognized by writers on the mathematical theory of elasticity. Nevertheless, for some occult reason the rigid attachment of angle iron stiffners to the web of the deep girder has been treated as immune from ths general law.

Failure of the thin unstiffened web of a deep rolled beam takes place by buckling in corrugations normal to the curves of ‡displacement in the compression zone and to the curves of ‡equi-potential in the tension zone of the web. Initial buckling takes place at the locus of greatest angular distortion of the metal, i. e., where the sum of the rhombic and trapezoidal deformation from horizontal shear and bending is a maximum. This locus has been determined graphically for the 15-inch I-beam of ten, twenty and thirty-foot lengths, page 22, Sec. II. With the increase in relative length this locus shifts away from the end. Thus for a central load on the 15-inch beam investigated, it was $L/16$ from the end with a length of 10 feet, $L/8$ from the end with a twenty-foot length and $L/5$ from the end with a length of thirty feet.

As the maximum unit compressive strain is one-half the maximum shear strain it may be readily determined for any specific case and a unit strip subject to that strain may be investigated for crippling by the solid column formula for fixed ends of Sec. IV, taking L as equivalent to the depth between flanges.

Stiffness or freedom from vibration requires sparing use of an unstiffened web in the railroad structure and its limitation to sixty thicknesses between flange angles with substantial end stiffners and a cross shearing stress not greater than four thousand pounds per square inch at the quarter point is a safe rough rule for shallow stringers.

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*Modern Frame Structures, pp. 147, Ed. 1909.

†For controversial discussion see Eng. News, Apr. 25, 1895, page 275, pages 339-340; Eng. News, Aug. 11, 1898, page 90; August 22, 1898, page 186; Dec. 22, 1898, page 400; May 19, 1898, page 322; June 9, 1898, page 311.

‡Note: Stress line analysis of 20″ I and development of curves of greatest displacement and equi-potential, Page 79-80, Sec. II.

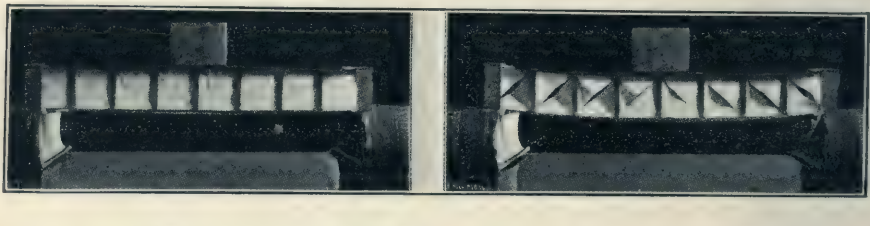
Because the internal shear strain (and its compressive component producing buckling) is a moment function, not directly proportional to the external shear force, any formula for web resistance to buckling based upon external shear force is essentially an approximation lacking in precision.

The trapezoidal component of shear distortion increases as the span length and decreases as the depth increases; the rhombic component increases as the ratio of the flange area divided by the longitudinal web area increases. Hence, for a given section the locus of the maximum shifts along the beam as the span increases in length relative to the depth.

8. The Joint Action of Web Plate and Stiffners was early investigated by Joseph H. Wilson and his conclusions stated in a paper on "Specifications for Iron Railroad Bridges", Trans. Am. Soc. C. E., 1885. His experiments with the model illustrated in the accompanying cut are described by him in an instructive manner as quoted from Engineering News, August 11, 1898.

> "When I went to Altoona on the Pennsylvania R. R., in 1860, I found in use on that road plate girder bridges, 50 and 60 ft. long, or longer, and 4 to 5 ft. deep, having only 3 16-in. webs, with vertical stiffners of cast iron, placed on each side and bolted together through the web by three bolts, these being the only connection. The stiffners hugged well up under the flanges of the girder and were spaced at intervals equal to the depth of the girder, more or less. These bridges were, without question, the best and stiffest type of structure on the road, were considered in every way the most reliable, and were carrying with safety any load that could, with the then existing rolling stock, be placed upon them. Some of them may be still in use on branch roads.
>
> "When, however, I came to design such structures and applied Rankine's rule for vertical web, 'considering that the shearing stress at the neutral axis is equivalent to a pull and a thrust of equal intensity, inclined opposite ways at 45°, and that the vertical web tends to give way by buckling under the thrust,' I found to my surprise that these bridges ought to have been utter failures.
>
> "The matter gave me considerable study, in the course

of which I made a model of a girder with a thin paper web, and I found at once that the web did not resist by compression and give way by buckling, but by tension. I made diagonal slits in the web, as you see in this model, converging downward towards the center. These being in the line of tension did not affect the strength in any way, as you may see by placing this model on supports at the ends, with the proper flange, as marked, up, and applying a heavy weight. When, however, I reversed the girder, with the other flange up, making these slits or cuts converge upward towards the center and placing the web in a condition that it could not resist by tension, the weakness of the girder was at once apparent, the web buckling up under compression. It was upon this practical solution of the problem that I formed my theory of the computation of web and stiffners, which I stated in my paper, and which for thin webs I still maintain.



"If there are no stiffners, Rankine's rule holds good and the case is one of a lattice girder with an infinite number of lattice pieces forming a continuous web.

"The increased thickness of web adopted in later years has not been due to theory but entirely to practical considerations, principally that of deterioration and rusting of the metal in thin sheets from the action of the coal smoke of engines and also to increase the bearing surface for rivets. Webs riveted in between angles have, of course, a great resistance from the hold which the angles have on them like the jaws of a vise, and in the early days there was not as much attention paid to the bearing surface of rivets as now. Shearing was the only matter considered. In those early days, also, engines burned wood, and coal-smoke with its sulphur fumes did not exist.

"When one comes to the use of a thick web and proposes to use it for all it is worth, the problem naturally becomes more complicated. A certain portion may be included in the flanges of the girder and often is, by those economically inclined; perhaps a portion with the stiffners; and then, if thick enough, it may also resist by compression as well as by tension approximating to a girder without stiffners. For my own part, I would prefer to make my calculations on the theory I have indicated, and having obtained the minimum dimensions for parts, then increase them as practical considerations from my experience, questions of wear and tear, etc., would make advisable."

From the truss analogy of Mr. Wilson, the practical rules for web resistance owe their origin. Regarded as a Pratt truss, the web as a tie is good for about seven-tenths its net section and if the vertical section of the web under the shear of the external force be figured at 10,000 lbs. per square inch, the diagonal tension will not exceed 15,000 lbs. per square inch of the A. R. E. A. specifications.

In 1888 the author observed a girder on the Jersey Central Railroad about six feet six deep stiffened at intervals about the depth of the web apart. In one panel the plate was buckled thru cooling stresses in rolling and had not been straightened in the fabricating shop. Under passage of heavy coal trains the buckle was observed to flatten or straighten out and it occurred to the writer to make a model girder with a symmetrically buckled web in order to measure the proportionality of the spring of the plate to the load applied. By the application of the load without impact vibration of the web was eliminated and satisfactory measurements made. See Trans. Eng. Soc. Western Pa., 1898. Trans. Am. Soc. C. E., August, 1902.

The method of thermo electric measurement of stress was applied to the web of the model girder and the bands of maximum stretch and squeeze along the neutral plane were located by the heating and cooling of the metal under the applied stress approximately as indicated by the figures A and B illustrating this effect by drawing lines closer or further apart to represent the relative intensities.

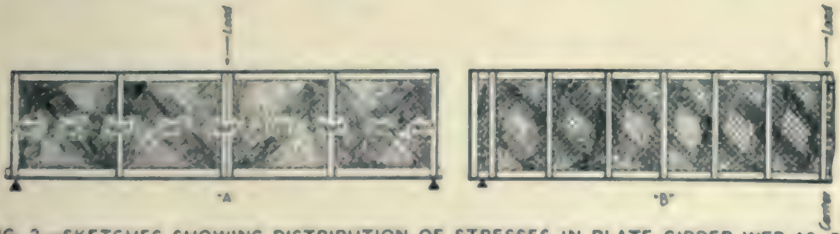


FIG 2—SKETCHES SHOWING DISTRIBUTION OF STRESSES IN PLATE-GIRDER WEB AS DETERMINED BY TESTS.

Fig. A represents the condition found with the stiffners the depth of the girder apart and Fig. B the modification caused by placing the stiffners half the depth apart.

With the thermal junction on the neutral axis of the girder attached to the web either near a stiffner or in the center of the panel, the galvanometer would indicate initial applications of the load, the deflection with the attachment near the stiffner being the reverse in direction to that when the junction was attached in the center of the panel, the wiring remaining the same.

The I-beam lever by which the load was applied to the experimental girder was also investigated but no galvonometer deflection was found when the load was applied, the junction being attached to the center of depth of the beam. As the load was increased from a few hundred pounds to twenty tons concentrated on the center of the girder with proportional galvanometer of heat and cold and spring of the buckled panel, the results are not open to question on the ground of deviation from elastic action occurring at the approach to incipient failure.

Referring to Fig. A, it may be noted that the lines of maximum compressive displacement are concentrated as it were in the dark area near the stiffners; (2) that they have changed in direction from 67½° being deflected toward that portion of the plate best able to resist compression by virtue of the lateral support of the stiffner angle; (3) that the path over which the maximum displacements act is but slightly longer than it would be were the section uniform with displacement at 67½° to the neutral axis of the girder at mid depth.

We may summarize these deductions in the statement that the load seeks the shortest course consistent with the more rigid path. Under tension, the strength of a member varies as its cross sectional area but under compression the form of the member is as important as its sectional area. For example, a 12 x ½ plate 10′0″ long is almost worthless as a column while the same sectional area in the form of two 7″ channels forms a strong and rigid strut. Thus the stiffner by its form more than by its mere sectional area renders the plate under and near it stronger and more rigid under compression than that portion more removed from it. The action of the stiffner in affecting the distribution of the stress in the web may be regarded as two fold: First, in rendering the plate in its vicinity better able to resist compressive displacement thru the lateral support it affords, thus causing an uneven distribution of the stress in the web as shown in the sketch, and second under favorable conditions it may relieve the web of a considerable portion of the compressive web stress. For the former action, few rivets are required but to act efficiently in the latter capacity where the stiffners are economically spaced they should be well riveted as was the case in the experimental girder.

The inefficiency of inclined stiffners compared to vertical may be established by the following consideration: If the stiffner is inclined at 45° and riveted to the web the compressive displacement of the material of the web is transmitted thru the rivets to the stiffner and a concentration of compression by Hooke's law would occur in the plate in belts following a Warren system instead of a Pratt. The path of greatest compressive displacement is thus increased in length over a vertical arrangement and heavier stiffners would be required while the detail would be inferior for the support of the flanges and questionable from the standpoint of counter stresses which are well provided for when the stiffners are applied vertically. The spacing of the stiffners should not exceed greatly the depth of the girder nor more than about 5′6″ for the heavy deck girder span.

Upon the assumption that the web stresses in the plate girder follow the law of distribution governing a beam of uniform section, the customary manner of determining the length of the cover plate is based. But the truss analogy of Mr. Wilson shows that this method gives too scant length for the upper flange plates and rather liberal lengths for lower flange. At least a panel excess should be allowed for the upper flange plate over that length which would render the variation of section proportional to the parabolic curve of moments from mid span to end.

The theoretical objection urged by our school men against the truss analogy is that horizontal shear thru the web is equilibrated at mid depth and constitutes the reaction to the flange stress. That mathematical theory shows such a state of shear is synonymous with principal tensile and compressive stresses at 45° to the plane of equilibration of shears; hence we must reject the experimental results of Turner's tests or revise, recast and find what is wrong with our mathematical theory.

The argument embodies a misconception of the significance of the theory and its misapplication. Change in temperature of the metal is caused by displacement of the molecules, the elastic cooling effect by stretch and the elastic heating by squeeze but the maximum stretch and squeeze are the joint product of principal stress and shear combined instead of the sole product of the principal stresses.

Sketch Figs. A and B merely locate zones or areas of temperature change. The springing of the buckled web locates the general direction of the maximum stretch. To determine the stress line mechanism refer to the stress line analysis of the 20″ rolled steel eye beam, page 80, Sec. II, noting that while the curves of principal tension and compression differ little from the rectangular section, concentration of the metal in the flanges causes a change in the shear stress area, a flattening of the curves of greatest shear and a corresponding change in the curves of greatest displacement and equi-potential, i.e., the direction lines of greatest squeeze and stretch.

Ribbing the web by vertical stiffners causes a further flattening of the curves of greatest vertical shear, a rotation of the curves of displacement (squeeze) of the compression zone toward the stiffner and a like rotation of the curves of equi-potential (squeeze) in the tension zone toward the stiffner. Therefore, by investigating the direction of the curves of greatest squeeze and stretch (bisecting the angle between the curves of greatest shear and principal stress by the mathematical theory) we find that the truss action found by experiment should have been a priori indicated by the elastic theory had that theory been heretofore fully understood and consistently applied.

9. The Flange Section may be proportioned from the moment of inertia of the net cross section, but computation is simplified by treating the flanges as truss chords having a lever equal to the distance between their centers of gravity and adding one-eighth the cross section of the web as effective flange area. This rule is derived as follows: Since the mean stress in the web (following the linear law) is half the maximum and the lever is two-thirds $d/2$ above or below the neutral plane, the resistance of the web would be as one-sixth its area concentrated in the flange were it not for the rivet holes. By taking an eighth of the area instead of one-sixth a fair allowance is made for the reduced section.

10. Riveting of Flange to Web must develop the horizontal shear necessary to the unitary action of the web and flange. If the flange be assumed to resist the entire moment, then it is argued that from the equation $Sdx = dM$, the shear between web and flange per linear inch of flange equals the vertical shear S divided by the distance between pitch lines of the two flanges in inches. Rivet spacing has usually been figured by this rule-of-thumb in the past.

*Some would reduce the shear to be thus provided for by one-sixth as an allowance for web resistance to moment where web area equals the area of the flanges and a proportionately smaller amount where its relative area is less.

But this practice is founded on a fundamental misunderstanding of beam theory, for it assumes the internal shearing strain is proportional to the external shearing force. At any given section the external shear force is balanced in part by the internal shear strain and in part by the vertical component of the principal stresses as is evident from a glance at the stress line mechanism of the 20″ beam, page 80, Sec. II.

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*C. W. Bryan. Modern Framed Structures.

External shear force gives rise to external bending moment which is exactly equal to and balanced by the internal resisting moments of the horizontal fibers at mid span or where the shear passes thru zero, and the ± displacement of the tension and compression flange are likewise zero. Regardless of the disposition of the load, whether uniform or concentrated the law of summation or increase of the ± elongation of the flanges is from zero near the center to a maximum at the end. Hence the shear between the web plate and flange resisted by the rivets increases from zero at this central neutral point to a maximum at the end.

This proposition may be regarded as a corollary to St. Venant's demonstration that the stress lines of a beam remain substantially unchanged in location and direction under any change of symmetrical load from uniform to a central concentration. (See page 81, Sec. II.)

Because the horizontal rhombic distortion of the web constitutes the reaction to the ± flange elongations, the shear between web and flange by Hooke's law is measured by the magnitude of this angular deformation which was termed "*h*" in the nomenclature of Sec. II and its maximum or end value was found in a simply supported girder for uniform loading as

$$\text{“}h\text{”} = 8\,F\,A\;/\;3\,t\,L\,F$$

in which F is the modulus of shear, A the area of flange at the center, t thickness of the web and L the length of span. Let q equal the unit shear stress at the end between web and flange and V the end shear and d the depth. Then

$$h = q/F = 8fA/3tLF = 2V/3td$$

Since $f = VL/4Ad$ therefore

$$q = 2V/3td$$

Let R equal the rivet pitch and B the allowed bearing value per square inch, which for enclosed bearings may be taken at 18,000 lbs. per square inch for medium steel; then

$$R = B/q$$

The double shear value of the rivet must of course be as great as the bearing value, but the latter always governs in the economically designed girder.

Accordingly, exact theory shows that for uniform load rivets may properly be spaced fifty percent further apart at the ends but the same at the quarter point as in ordinary practice. Since the angle "*h*" varies as a third degree curve whose area is five-eighths its rectangle, the excess riveting is to that required by exact theory as 2/3 x 5/8 : 1/2 :: 5 : 6, a twenty percent excess.

For a case of two heavy concentrated loads at one sixth *L* from the end, the values would be slightly in excess by the common theory, but at the quarter point deficient. Our erroneous practice involves in ordinary cases a material waste of rivets, while in case of symmetrical concentrated loads near the end, a twenty to thirty percent deficiency at the quarter point may result. For locomotive loading greater waste than that of rivets results from the use of excessively thick plates for bearing values in keeping with false theory and the adoption of a depth too shallow for economy increasing the weight and decreasing the stiffness of the design.

For the cantilever girder the common theory results in the closest rivet spacing at the support where the fewest rivets are needed and the fewest rivets at the end where the most are theoretically required and the same statement applies to the cantilever portion of the continuous beam.

Because the rhombic shear distortion of the web is zero at the support of the restrained beam, the same web is capable of resisting a much greater external shear force than when the girder is simply supported. Fewer rivets are required and the minimum thickness of web plates may be used that permanence against corrosion will permit. In general the rivet spacing should not exceed about ten thicknesses of the flange metal nor over four inches alternate spacing in the flanges of deck girder spans.

11. The Position of the Load for the Maximum Horizontal Shear on the Rivets between the web and the flange is that position of the loading which produces the maximum moment because the shear between the web and the flange is the reaction to the moment stresses.

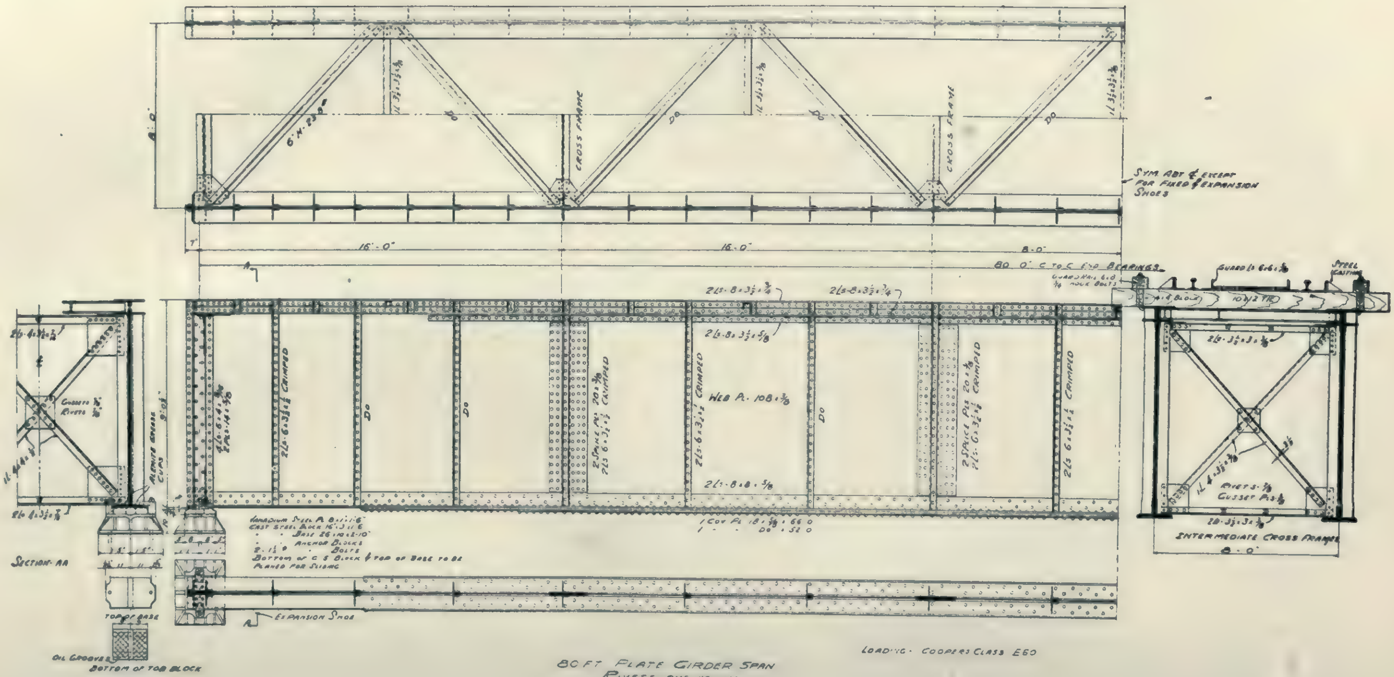
The customary practice, having determined the reaction, is to treat the reaction as tho it were the particular vertical shear which produces the greatest stress on the rivets.

Now the reaction for engine loading considerably exceeds the shear which produces the greatest moment by fifteen to twenty-five percent which together with the error of the common theory is partly offset for the girder by the excessive allowance of 24,000 lbs. per square inch in bearing on the web instead of 18,000 lbs., which is permissible. But for the truss the 24,000 lbs. bearing per square inch of our A. R. E. A. specification leads to undue weakness of pin plates and truss joints, as indicated by Howard's column tests discussed in Section IV.

12. Enclosed Bearing and One Side Bearing Rivets: Where the rivet bearing is enclosed and the rivets passing thru the web plate of the girder are in double shear and as the bending of the rivets is symmetrical, they develop a higher bearing value than where the shear is single and the rivet tends to rotate. This rotating tendency should be allowed for not only by a lower bearing value such as 12,000 lbs. per square inch but also by an increase of the section of the spliced material an eighth greater than the section spliced on which the state of the rivets is that of enclosed bearing as against one side bearing on the splice bars. Thus butt splice bars should have a combined cross section an eighth greater than the part spliced as recommended by Unwin in his work on machine design.

13. Elastic Working Stress on the Rivet is determined by combined direct tension due to the shrinkage in cooling, bending and shear due to its load. The direct tension may be treated as a constant for rivets of different sizes. The bending is relatively less as the ratio of the thicknesses of the part connected divided by the diameter of the rivet decreases. In other words, bending resistance of the rivet increases as the cube of its diameter while the shearing resistance of the rivet increases as the square of its diameter and for this reason larger rivets are more satisfactory for field connections than smaller ones. In the railroad bridge floor and stringer connections 1-1/8 inch rivets are the smallest diameter which should in general be used. In the old days when rivets had to be driven by hand 7/8 inch diameter was in keeping with the best results to be secured by this method of driving, but with pneumatic hammers in general use today, the larger diameters of rivets are readily driven and better results may accordingly be secured by their use.

14. Smooth Top Flanges for Deck Girders are preferred by the Maintenance of Way Engineer because it obviates the excessive notching down of ties and cutting for rivet heads. An example of such design for an E60 loading is shown in the accompanying cut with lateral bracing confined to the compression flange with ample section provided for its rigidity, while the bottom flange is braced at intervals by cross frames.



80 FT PLATE GIRDER SPAN
RIVETS - ONE INCH UN
SCALE ¾ IN = 1 FT

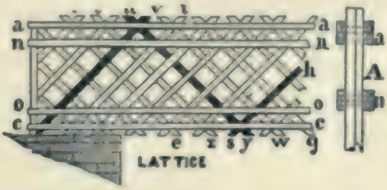
CHAPTER XXV

ECONOMIC TYPES, HEIGHT AND PANEL LENGTH OF SIMPLE SPAN THRU RAILROAD BRIDGES

PIN VS. RIVETED CONSTRUCTION

1. **Historical:** As longer and longer plate girder spans were constructed, rolling mill limitations fixed the lengthwise dimensions of the web plates vertically requiring so much metal in the splices that lack of economy resulted and the open web replaced the solid web of the long span Brittania[*] tubular bridge types. The metal in the open web is concentrated in the natural lines of greatest displacement of the stiffened web as shown by the experiment with the model girders and we have the lattice girder of single, double or multiple intersection, as the case may be.

Notable in early timber construction of bridges is the Towne lattice (1820) made of planks crossing each other at right angles in the web pinned or bolted at intersections with chords of heavier timber at the top and bottom as shown in the figure. The unit system of the multiple webbing is illustrated by the black plank (after Trautwine) the chords being common to all the web triangulations.



This type in iron has been followed in many continental bridges and some in this country and Australia. The relatively thin plank gave trouble from warping and twisting and the [*]Burr type (1804) of timber construction replaced the Towne lattice. Open single triangulation with counter stresses relieved by an arch of timber bolted to the frame as illustrated in the cut, constituted such an improvement that Burr trusses over a hundred years old are still substantial and giving good service in parts of Pennsylvania and the East.

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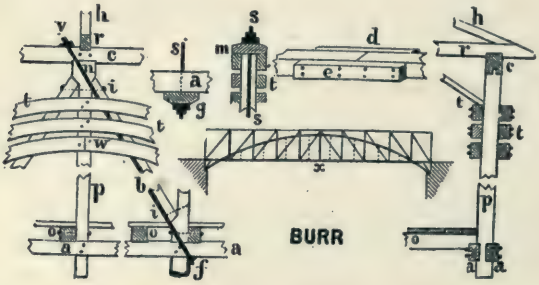
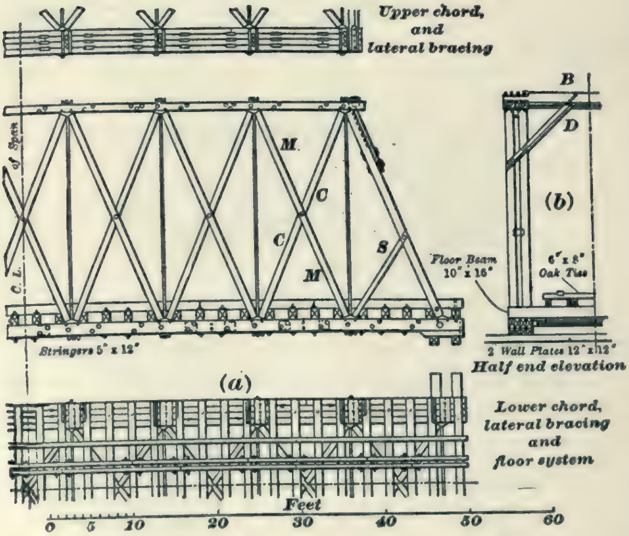
[*] Notable Tubular Bridges designed by George Stephenson are the Conway over Menai Straits, 400-foot span, 1848; the Brittania two side spans, 230 by 23 feet deep, 2 central spans, 400 feet long with 30-foot depth girders, 15'0" on centers; Bratherton over the Aire and the Victoria over the St. Lawrence River. From the facts noted it will be seen that the early riveted plate girder bridges had depths up to 30 feet, while today we think of 11 feet as the limit of height.

[*] Burr truss over the Lehigh River at Bethlehem, Pa., built 1816, was not replaced until over a century later.



As iron became cheaper the vertical timber tension members of the Burr truss were replaced by the iron tie rods of the Howe improvement (1840) the arch for counter strain being replaced by Howe with light timber counter braces or true truss members.



The Warren Truss was evolved from the Towne by the elimination of the multiple cross lacing of web members simplifying the frame to a SINGLE INTERSECTION.

Progress has been toward simplification toward determinate types and economic reduction of the multiplicity of joints and connections of older work. In iron for the lighter structures the tie bar was cheaper than the strut, hence the idea of making the web a maximum of tension member and a minimum of compression member led to the reversal of the direction of the diagonal main members of the Howe truss, i. e., vertical posts and diagonal eye bars typical of the Pratt truss (1844).

In tracing the chronology of the bridge types, the rapidity of their introduction rather than the date of invention governs from the practical standpoint as outlined.

The construction of metal bridges in the wake of the old timber bridge was influenced by the practice of timber construction just as the concrete building in its early days was influenced by the timber and steel constructions which preceded it.

The first iron truss bridge in the United States was built in 1840 over the Erie Cana' by Earl Trumbull. Some members were cast and some wrought iron. In 1840, Squire Whipple built his first bow spring truss and published a work on stresses in bridge members in 1847. His first double intersection truss 146 feet in span was constructed for the Rensselear and Saratoga Railroad. By 1850 the Pratt trusses were made entirely of metal. Lattice trusses entirely of wrought iron were built for the New York Central Railroad in 1859. In 1861 J. H. Linville introduced forged bars and wrought iron web members using cast iron for the upper chords.

The first Baltimore type truss was designed in 1871 by the Bridge Department of the Pennsylvania Railroad. In 1870 Parker modified the Pratt truss using inclined upper chords, an improvement subsequently adopted with the Baltimore type.

2. The Truss Principle is that of resistance to deformation by axial strain along the length of the truss member.

As the triangle is the only geometric figure that is incapable of change of configuration without change of the length of its sides, the truss is inherently a combination of triangular frames. Its investigation from the economic standpoint as in the beam consists of determining the economic depth and the form and economic number of triangular units combined in its make-up.

3. Economic Depth: We have noted in plate girder stringers that the depth of the web should preferably be not less than one-seventh of the span length. When the web is made open its weight is reduced and for parallel chords economic depth in terms of span length is increased to keep the economic balance between the weight of web and the weight of the chord. But as the span length is increased the relative depth of the wind or lateral truss decreases and the angle of inclination of the diagonal members becomes very flat if the panels are as long as economic main truss panels should be and the conflict between the economic requirements of the lateral truss and the vertical truss must be compromised. With the pin connected type of bridge this compromise has been effected in some designs by a subdivision of the Baltimore type, the sub-vertical with the plain Warren or by double intersection, but as the compromise is to the detriment of economy of the main truss, the best arrangement is that of partial independence of the paneling of the main system and the lateral system. Such independence is readily effected with the riveted type of structure and impracticable with the pin connected type. This constitutes the first count in favor of the riveted bridge as against the pin connected type and which will be more fully discussed in connection with practical design of economic long span structures.

4. Relative Economy of the Pin vs. Riveted Type of Structure was a subject of controversy twenty-five to thirty-five years ago. Continental engineers favored the riveted structure, claiming for it greater rigidity without sacrifice of economy. American engineers maintained that the pin connected structure was easier to erect, weighed less and while not as free from vibration as the riveted structures, its total deflection on account of greater depth was less and that there was a degree of determinateness and certainty about the pin connected joint which was lacking in the riveted joint.

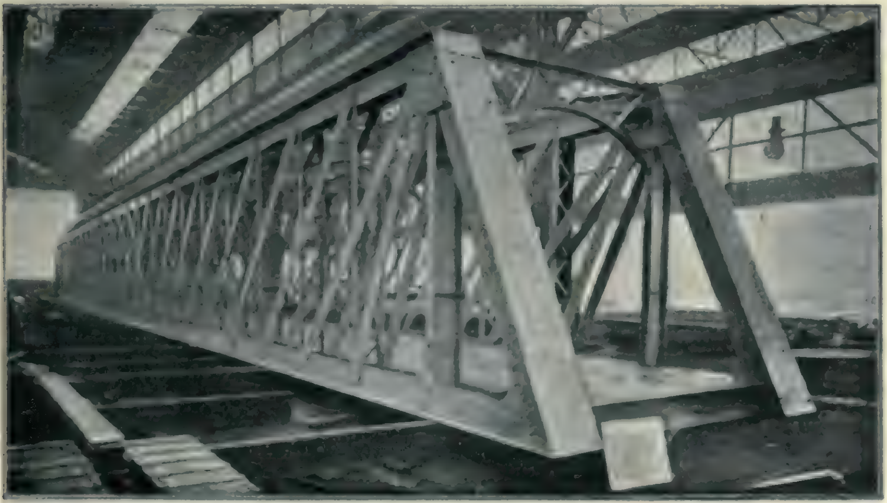
For comparative purposes we illustrate a riveted bridge reproduced from the handbook of Dorman Long Company, Ltd., of Middlesbrough, England, showing a 200-foot span English bridge and an American structure from Ketchum's Handbook representing

Because the mechanical analysis of stress in a truss frame has been so fully treated by many authors, the student will be here referred to Ritter's work (1879), on Iron Bridges and Roofs, Trans. by H. R. Sankey, published by Spon & Co., New York and London, and to Researches in Graphic Statics by H. T. Eddy, 1878, as best covering the ground not treated in the present investigation. In fact the discussion of trusses in Trautwine or American Engineers Pocketbook covers the elementary principles employed in the present investigation. Ritter's treatment of moments and the determination of truss forms meeting arbitrarily fixed maximum stress relations it is thought will prove of value in conveying a broader comprehension of the subject.

a riveted bridge designed by Mr. Hedrick of 160-foot span and a pin connected bridge designed by Mr. Modjeski of 150-foot span.

Compare the floor beam connections: In the riveted structure the beams are square ended. In the pin structure, the web is cut away to clear the pin and arrange for the lateral connection expensive shop work and heavy gusset plates are required which are unnecessary with the riveted structure. The 150-foot pin span was designed for E50 live load and weighs 315,000 lbs., the riveted 160-foot span was designed for E45 loading and weighs 303,000 lbs., since its length is ten feet greater, it appears from the relative weights that there is some difference in favor of the riveted structure. While the English design lacks economic depth, has too many panels for economy, its deficiency is that of degree only as neither of the designs compared present economic panel lengths for their respective loads and spans.

The shop work for the same panel length is less for the riveted bridge than it is for the pin connected type shown. A true comparison of the relative merits of the types involves investigation of the respective structures arranged with the most economic panel lengths, heights, and details for each.



English Type Riveted Span

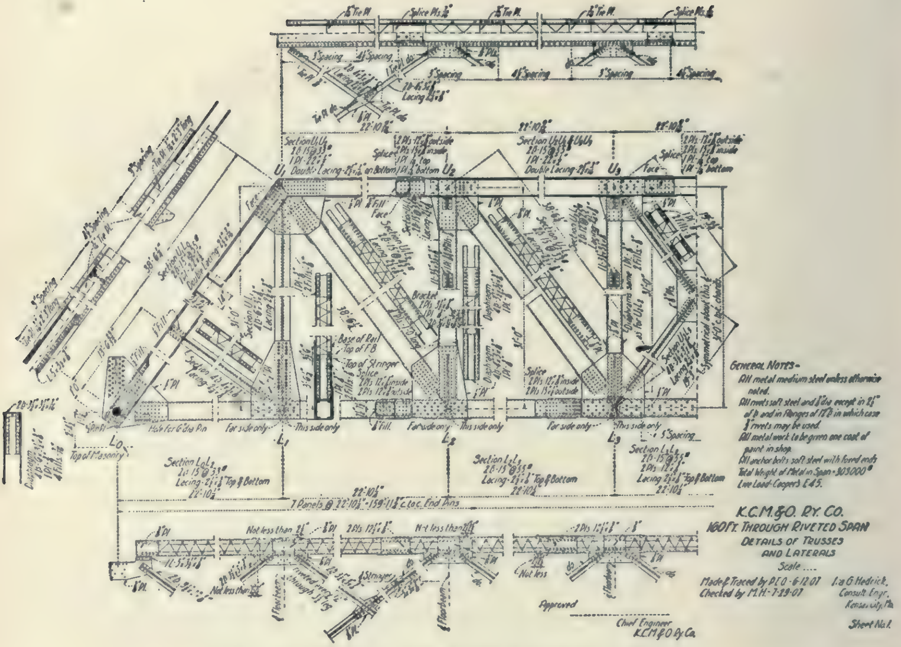

Splice Pls.
Section U1U2
Section U2U3 & U3U4
Double Lacing
U1
U2
U3
L0
L1
L2
L3
Section L0L2
Section L2L3
Section L3L4
Base of Rail
Top of F.B.
Top of Stringer
Bracket
Diaphragm
Splice
Face
For side only
This side only
Hole for 6" dia. Pin
Top of Masonry
7 Panels @ 22'-10½" = 159'-11½" c. to c. End Pins
Not less than
Stringer
Floorbeam
General Notes –
All metal medium steel unless otherwise noted.
All rivets soft steel and ⅞" dia. except in 2½" of ⅞ and in flanges of 12" ⅞ in which case ¾" rivets may be used.
All metal work to be given one coat of paint in shop.
All anchor bolts soft steel with forged ends.
Total Weight of Metal in Span = 303000#
Live Load - Cooper's E 45.
K.C.M.&O. RY. CO.
160 FT. THROUGH RIVETED SPAN
DETAILS OF TRUSSES AND LATERALS
Scale ....
Made & Traced by P.E.O. - 6-12-07
Checked by M.H. - 7-29-07
Ira G. Hedrick, Consult. Engr., Kansas City, Mo.
Sheet No. 1.
Approved
Chief Engineer K.C.M.&O. Ry. Co.

FIG. 39. SINGLE TRACK THROUGH RIVETED TRUSS RAILWAY BRIDGE.

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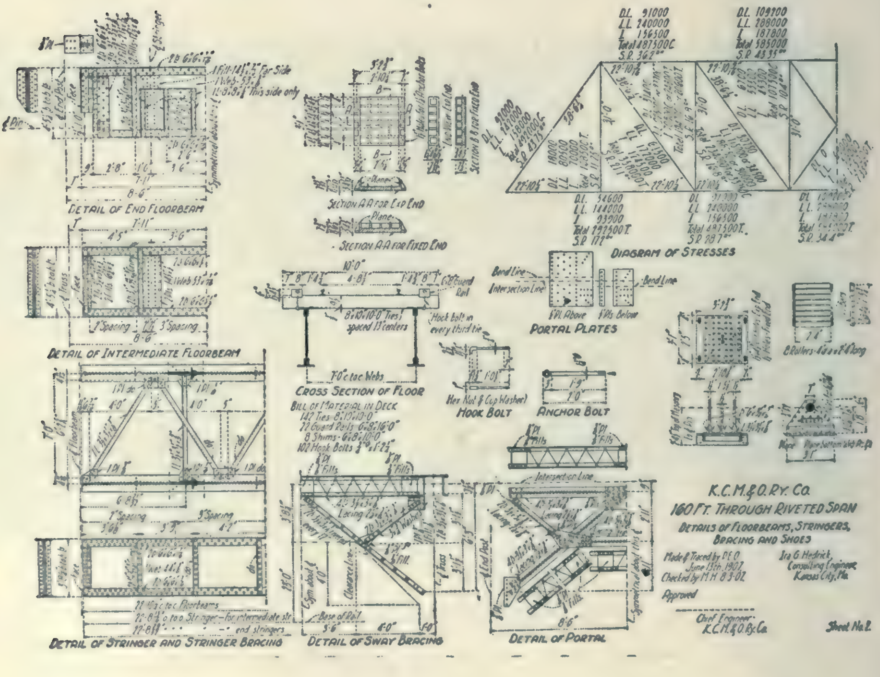


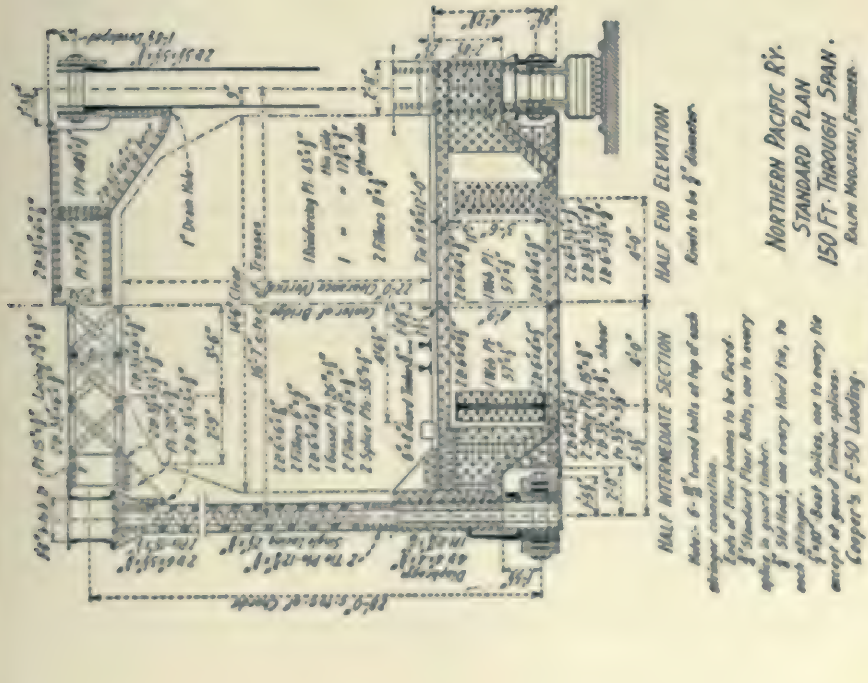
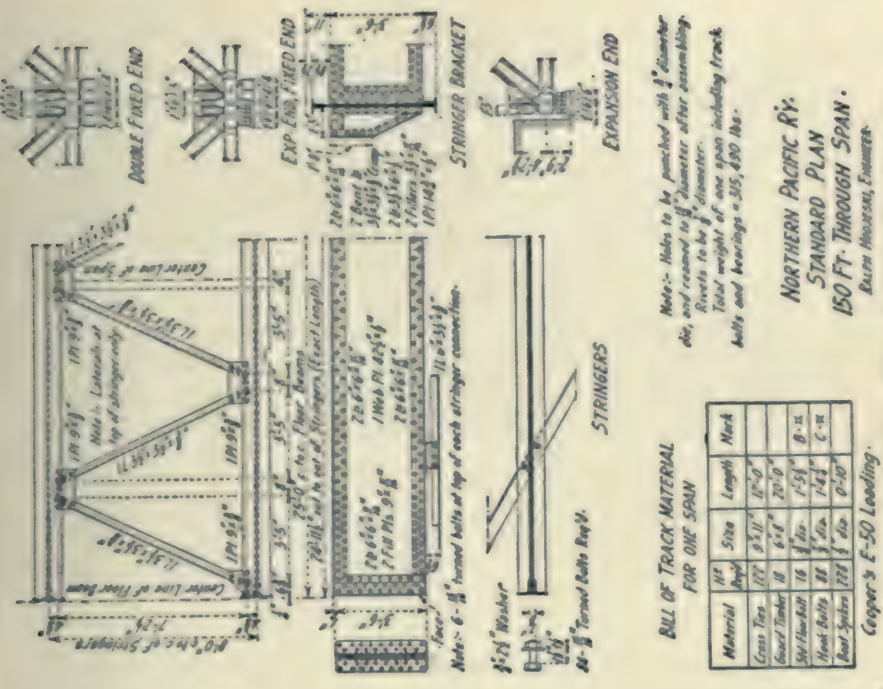
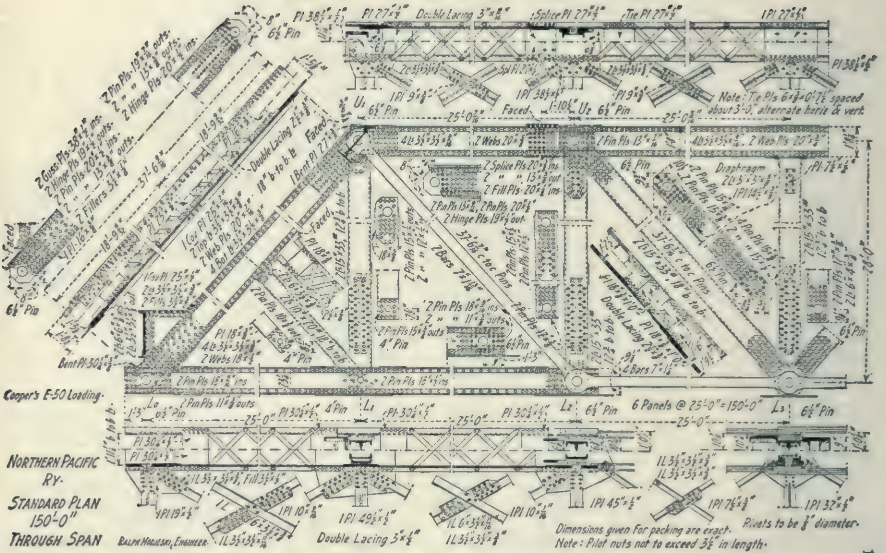

HALF INTERMEDIATE SECTION
HALF END ELEVATION
Center of Bridge
22'-0" Clearance (Horizontal)
Transverse
1" Drain Hole
28'-0" C. to C. of Chords
Single Lacing
Diaphragm
Note:- 6-⅞" turned bolts at top of each stringer connection.
Ends of floor beams to be faced.
⅞" Standard Floor Bolts, one to every spike in guard timber.
⅞" Std. Hook, one every third tie, to each stringer.
⅝"x10" Boat Spikes, one to every tie except at guard timber splices.
Cooper's E-50 Loading.
Rivets to be ⅞" diameter.
NORTHERN PACIFIC RY.
STANDARD PLAN
150 FT. THROUGH SPAN.
RALPH MODJESKI, ENGINEER.

FIG. 45. SINGLE TRACK THROUGH PIN-CONNECTED RAILWAY BRIDGE.


DOUBLE FIXED END
EXP. END, FIXED END
STRINGER BRACKET
EXPANSION END
Center Line of Span
Center Line of Floor Beam
Note:- Laterals at top of stringers only.
8'-0" C. to C. of Stringers
STRINGERS
Note:- 6-⅞" turned bolts at top of each stringer connection.
⅞" Turned Bolts Req'd.
Note:- Holes to be punched with ⅞" diameter die, and reamed to 1 1/16" diameter after assembling.
Rivets to be ⅞" diameter.
Total weight of one span including track bolts and bearings = 315,490 lbs.
BILL OF TRACK MATERIAL
FOR ONE SPAN
Cooper's E-50 Loading.
NORTHERN PACIFIC RY.
STANDARD PLAN
150 FT. THROUGH SPAN.
RALPH MODJESKI, ENGINEER.

Material	No. Req'd.	Size	Length	Mark
Cross Ties	177	9"x11"	12'-0"	
Guard Timber	18	6"x8"	20'-0"	
Std. Floor Bolt	16	⅞" dia.	1'-5⅝"	B·X
Hook Bolts	88	⅞" dia.	1'-4¾"	C·X
Boat Spikes	728	⅝" dia.	0'-10"	

FIG. 46. SINGLE TRACK THROUGH PIN-CONNECTED RAILWAY BRIDGE.


6½" Pin
Double Lacing 3"×⅜"
U₁
U₂
6 Panels @ 25'-0"=150'-0"
25'-0"
L₀
L₁
L₂
L₃
6½" Pin
Diaphragm
Note: Tie Pls 6×⅜×0'-7½" spaced about 3'-0" alternate horiz & vert.
Cooper's E-50 Loading.
NORTHERN PACIFIC RY.
STANDARD PLAN
150'-0"
THROUGH SPAN
RALPH MODJESKI, ENGINEER.
Double Lacing 3"×½"
Dimensions given for packing are exact.
Note: Pilot nuts not to exceed 3½" in length.
Rivets to be ⅞" diameter.
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SINGLE TRACK THROUGH PIN-CONNECTED RAILWAY BRIDGE.

5. Investigation of Economic Panel Length is simplified by the fact that the amount of the tension members equals the amount of compression members measured by the summation of the products of the strain times the length of the members for any truss regardless of the number of panels. Therefore, the products of the strain times the length of the members, i. e., the strain area is a determinant of relative economy as the number of panels are increased one at a time.

In Section I, page 16, Chapter I, Art. 21, the principle of conservation of maximum and minimum fiber stress areas in an elastic solid was demonstrated. Analogous to this principle in the elastic solid we find in the truss frame a like conservation of tensile and compressive strain areas as may be seen in the computation in the following article. Having investigated the law of variation of truss weight with the variation of depth and panel length and found the economic proportions for the truss we may proceed to investigate the divergent weights of floor for different lengths of panel and number of floor beams involved, completing the analysis.

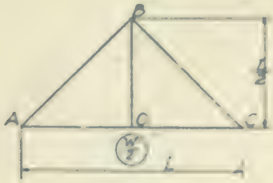
6. The Strain Area Coefficient measures the result of the investigation suggested in Art. 4. The most convenient unit is $W = wL$, the total load on the span, as the unit measure of strain. The depth is conveniently taken in fractional parts or decimals of

L, the span length. Then the coefficient embodies the weight or load, the span length, the depth of the truss and constitutes a suitable unit for the investigation of economic proportions and relations.

The particular advantage of selecting the weight unit as the product of the uniform load times the length of the span for comparative purposes arises from the fact that a different percentage of the unit W is carried by the truss with different divisions of the truss in panel lengths. Thus, in a two-panel truss but half of W enters into the truss as a determinant of strain and section; when there are three panels two-thirds W functions in producing stress and strain in the truss; when there are four panels, seventy-five percent of W functions in this wise; when there are five panels, eighty percent, etc.

The unit thus takes into consideration the amount of weight which enters into the determination of the section of the truss automatically. By measuring the strain length in terms of L, the span, the depth as a fraction of L enters into the unit. So that we have the manner of distribution of the load in its concentrations at the panel points, the depth of the truss, the inclination and the articulation of all of the elements except minor details of the floor system which may be readily investigated or approximated when we know the economic relations of the main truss sections.

Problem (a)



Given a triangular truss of two panels ABC. As the height equals the panel length Tan $\Theta = 1$. Sec. $\Theta = 1.414$. Hence

Bottom chord strain area $= W/4 \times 1 \times L =$ $WL/4$
Rafter strain area $= W/4 \times 1.414 \times .707\,L \times 2 =$ $WL/2$
Hanger strain area $= W/2 \times L/2 =$ $WL/4$

Therefore, the sum of the individual strain areas equals Σ S. A. the coefficient wanted, equals unity in terms of WL. If the height be increased to $0.6L$ then Σ S. A. $= 1.015\,WL$. If it be decreased to $0.4L$ then the coefficient increases to $1.0245\,WL$. Therefore $L/2$ is approximately the economic height.

7. **The Economic Depth** was found for the plate girder when the weight of the flange material resisting horizontal stresses of moment equals the weight of the material which resists the vertical forces of external shear.

In the triangular truss of the preceding problem it seems at first difficult to apply this rule because the rafters or inclined members resist both horizontal stresses of bending and shear of vertical forces. However, we know that the material of the rafters resists equally in this case horizontal and vertical forces then should half the weight of the rafters plus the weight of the hanger represent web material and the other half the weight of the rafters and the bottom tie represent the chords or moment resisting members and the criterion of economic depth is in this instance when half the weight of the rafters plus the hanger equals exactly the weight of the horizontal chord member plus half the weight of the rafters as measured by the strain area.

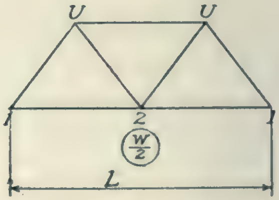
A depth of $L/2$ exactly fulfills this condition and no other depth ratio can do so. By applying this determinant of the economic depth, the depth which will fulfill the criterion almost exactly may be approximated from the first computation.

8. **Determination of Economic Panel Length** is simplified by determining the strain area coefficients for different frames and a sufficient variety of panel lengths. For example if the span be 150 feet, for a heavy loading we may proceed to investigate frames of three and four panel lengths.

Starting with two panels and a depth of four-tenths L, the strain area coefficient is found to be .8644; and examining for shear and moment strain areas substantial equality of the two is found for this depth. But 40 feet in depth for 100-foot span is somewhat excessive and the strain area coefficient for a depth one-third of L may be figured: For this depth Tan $\Theta = .75$ Sec. $\Theta = 1.25$

Bottom chord 1–1 = $.1875\ L\ W$
Top chord $u\ u$ = $.1875\ L\ W$
Diagonals $2\ (u{-}2)$ = $.5207\ L\ W$
Total = $.8957\ L\ W$

Thus the depth may be reduced quite materially from the economic height with but a small change in the coefficient.



For more than two panels the condition of partial (live) load is to be considered. Since the impact stress and allowance therefor is greater for the web member than for the chords we may closely approximate the practical conditions of the railroad bridge by treating the fraction of W constituting the panel load as a live load for the entire web system.

The accompanying figure shows the variation of the economic depth ratio and strain area coefficient with variation of the number of panels from two to twelve in span length for parallel chord Warren and Pratt truss diagrams.

The minimum strain area diagram illustrating the depth ratios gives a clear idea of the variation of the angularity of triangulation with a fixed type and different numbers of panels.

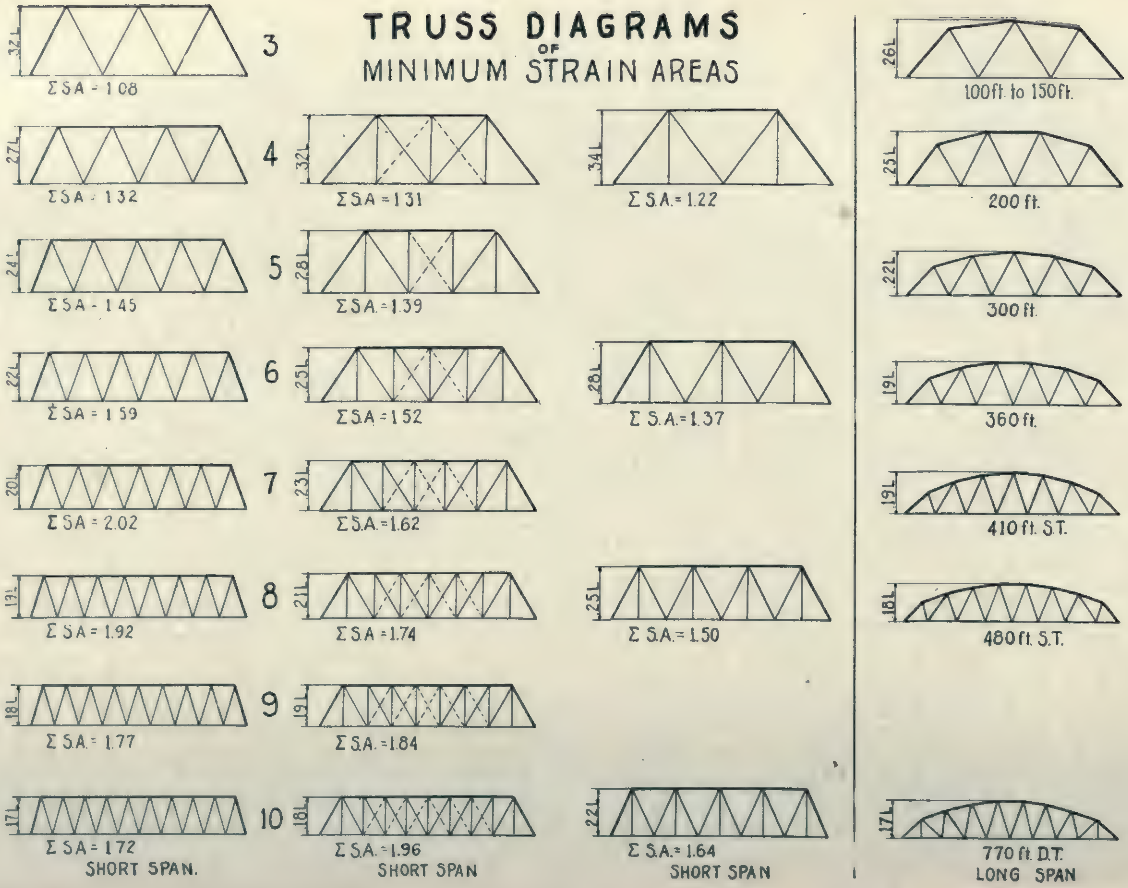
9. The Economic Panel Length is indicated by the strain area coefficient as an index of economy since the truss weight will increase or decrease approximately as these coefficients.

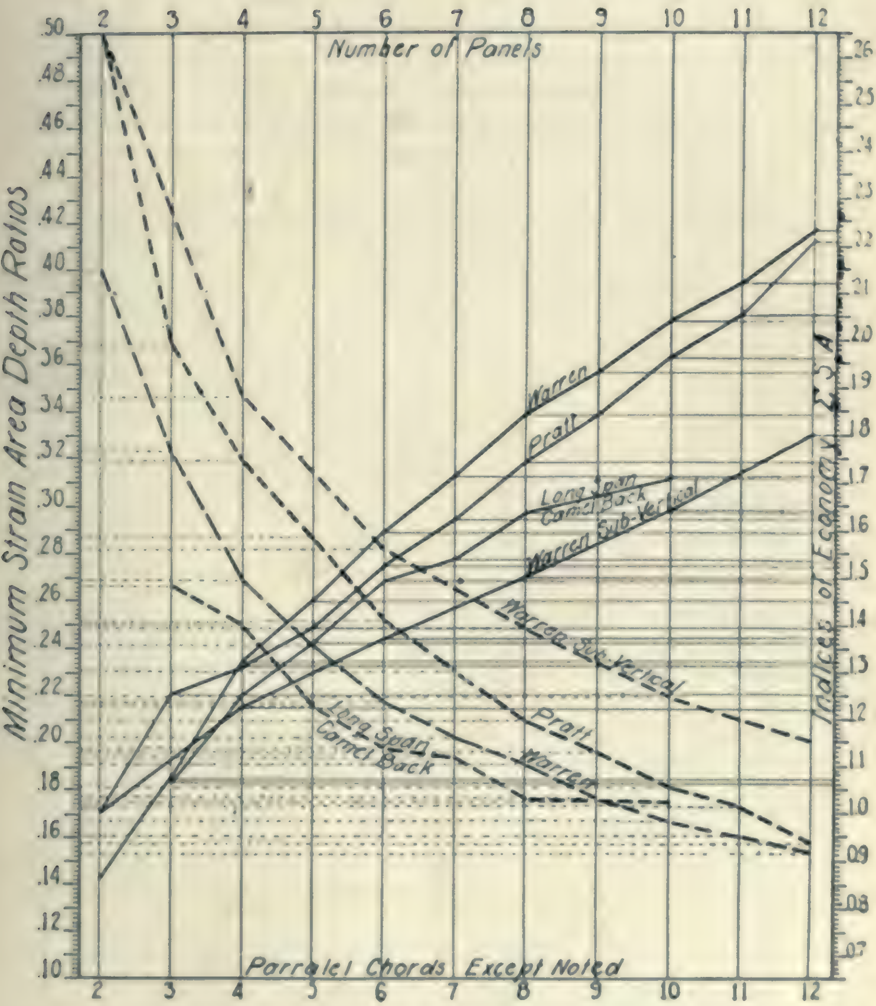
Thus the strain area coefficient for a three panel 150-foot span is 1.1, for a six panel, 1.6, nearly an increase of forty-five percent. But for the whole bridge floor weight is one-third and the economic saving will be a trifle less than thirty percent of the total figured in percent of the lighter or more economical truss or twenty percent or more as a reduction in weight of the heavier design.

Doubling the panel length of ordinary practice is a simple prescription for so great a saving, but there are other essentials to make it workable. On the debit side of the ledger the stringer moment and impact triple as the length of the panel in the 150 foot span suggested is doubled. On the credit side with three panels there are four stringer connections with 800,000 lbs. shear plus impact — 480,000 — 1,280,000 lbs. to provide for against 1,050,000 plus 900,000 impact = 1,950,000 lbs. to provide for in connections and field rivets with six panels. With three panels there are four beam connections 800,000 shear plus impact 480,000 against 1,950,000 with ten beam connections of the six panel truss, i. e., the field riveting in the floor is two-thirds as great with three panels as with six. There are six stringers for shop work with the long panels against 12 with short and two floor beams against five. With a three panel truss there are but five main joints against 12 with the six panel truss.

By making the stringers continuous, shortening the end panel and lengthening the center, the moment may be reduced and the stiffness increased largely by increasing the depth of the web plate in keeping with continuity and the increase in span length. The stringer weights for the longer panel will then exceed those for the shorter by the increase in web weight which is off-set by the saving in floor beams and connections.

TRUSS DIAGRAMS OF MINIMUM STRAIN AREAS




Number of Panels
2
3
4
5
6
7
8
9
10
11
12
Minimum Strain Area Depth Ratios
.50
.48
.46
.44
.42
.40
.38
.36
.34
.32
.30
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
Indices of Economy ΣSA
.26
.25
.24
.23
.22
.21
.20
.19
.18
.17
.16
.15
.14
.13
.12
.11
.10
.09
.08
.07
Warren
Pratt
Long Span Camel Back
Warren Sub-Vertical
Warren Sub-Vertical
Pratt
Long Span Camel Back
Warren
Parralel Chords Except Noted

Those who have been responsible for the maintenance of old overloaded bridges have had to patch up connection angles, split down the root for the lack of continuity of top flange or have had to cut out and redrive many rivets in too shallow bridge floors will best appreciate the practical merit of the suggested innovation.

10. The Relative Vertical Rigidity of the truss is measured by the strain area coefficient.

Comparing a three panel and a six panel 150-foot span of economic depth, the three panel is nearly one and a half times as stiff as the six panel since their respective strain area coefficients are as 1.1 : 1.6.

Maximum stiffness and truss economy thus go hand in hand because the strain area is a minimum for the economic depth.

11. The Lateral Stiffness of the single track railroad bridge is satisfactory or deficient dependent largely upon the use of stiff riveted bracing and a scientific triangulation of the lateral truss system. In a vertical Pratt truss no one would think of inclining the ties to as flat an angle as would result from a twenty-five or thirty foot panel length for a depth as shallow as seventeen feet. Nevertheless, just that kind of triangulation is the rule rather than the exception in the floor laterals of our thru railway bridges.

With nine lateral panels in a 150-foot span, the strain area for 17-foot center to center trusses gives a coefficient for moving load of .19; for six (25 foot) panels .312; for five (30) foot panels it is .356. Therefore, nearly double the metal is required for indentity of panel dimension of the vertical and lateral webbing that is required with independent paneling. Moreover, as far as the part contributed by the diagonals to the lateral deflection the stiffness is twice as great with the better triangulation of shorter lateral panel lengths.

Because independence of lateral and main truss triangulation is impracticable with the pin connected eye bar chord, such a chord is out of its element in the simply supported railroad bridge structure.

12. Combined Resistance or Coordinated Unitary Lateral Action of all the members of a floor system scientifically arranged is productive of the greatest possible rigidity. Thus with a favorable lateral triangulation the laterals should be so substantially riveted to the stringers that the stringers and riveted chords function as a part of the lateral truss. In the 150-foot span just discussed, because the section of the stringer is about twice as great as that of the chord, while its lever is half as great, the lateral strain in the chord may be reduced by half and the amplitude of the vibtations from train impact in a greater ratio.

The wood floor, rails, steel and wood guards can be made to contribute their approximate share by inserting between the 8" x 10" ties a 4"x4"x10" long spiked vertically to the end of the tie. Under derailment with this detail, no bunching up of ties can occur moreover with in and out steel guard angles of 6"x6"x5/8" metal well fastened, a high degree of lateral rigidity is presented by this wood and steel combination decking adding its proportionate share to the combined rigidity of the entire floor system.

13. The Lateral Rigidity of the Main Trusses of the thru bridge depends upon the joint rigidity of the top and bottom bracing, the portals, sway frames and the articulation of the main truss members.

Stiff bracing is the first essential for rigidity and two or three lateral system panel divisions should be made to one of main truss panels, because the depth in the sense of horizontal distance between chords is relatively but half or a third as great as the height of the main truss at mid span. As in the main truss single intersection for top laterals is more economical than double intersection and nothing is to be gained in the top system by symmetry as is the case with the bottom lateral arrangement. Riveted articulation is far preferable to the pin joint.

In the portals the breadth of the end post or its cover plate determines or limits the sway of the upper portion of the truss. Rigid restraint of the end post at the base thru the height that the depth of the stringer permits quadruples the stiffness as compared to the condition of posts unrestrained at the base. In like manner the efficiency of intermediate sway frames is determined by the transverse breadth of the posts to which the frame attaches and these dimensions should be worked out with the foregoing considerations in mind.

14. The Arched Top Chord is the Most Economic Truss Configuration because it takes advantage of the diverse laws of increase in magnitude of the shear and moment which it is called upon to resist. Thus the moment increases approximately as the ordinates of a parabola from the end toward the center and the arched top chord gives the maximum depth or lever arm for resisting moment at mid span.

Conversely the vertical shear increases from mid span toward the support. Hence the heaviest web members are toward the support and these are shortened and reduced in weight by curving the top chord downward.

Moreover, by its inclination the arched chord functions in a dual capacity, the horizontal component of its stress may be regarded as resisting moment proper and its vertical component as resisting shear so that the cross section of all the web members are economically reduced by this action.

15. The Parabolic Arched Chord presents properties most instructive to the student of economic design. Because the force polygon of a uniform load is parabolic, the web strains for this particular loading are reduced in function to that of a hanger carrying the weight to the supporting arch. The bottom chord acts as a tie with uniform strain thruout. The top chord stress increases from the center toward the support.

Let D equal the depth, L the span and W the total load uniformly distributed. Then $WL/8$ equals the maximum moment at mid span and $WL/8D$ equals the bottom chord stress and its strain area is $WL^2/8D$, which for equilibrium must be balanced by the horizontal component of the top chord strain area for each increment of span length from the end toward the center because there is no diagonal component under uniform load in the web stress.

Now the vertical component of the arched chord at the support equals the reaction equals $W/2$ and the variation in magnitude of this component between the support and mid span is proportional to the tangent offsets of the curve from the center to the ends, and the mean of these being $D/3$ from the properties of the parabola, the vertical component of the arched chord strain area is $W/2$ times $DL/3$, equals $WLD/6$. Now the strain area of the sum of the hangers is obviously $2WD/3$ and for a minimum

$WLD/6 + 2WD/3 = WL^2/4D$........................(a)

Solving $D = \sqrt{3L^2/(2L + 8)} = .546L$..............$(b)$

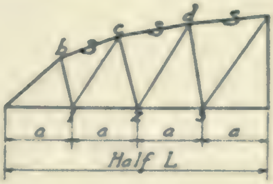
In the left of equation (a) since D is substantially $\frac{1}{2}L$ the second term representing the hangers is $4/7$ of the left member of the equation and the reduction for the value of D for live load treated in the previous example would approximate $D = .37L$.

But such relative height at the center is extreme or top heavy from the practical standpoint and deficient for the portal at the hip and that form of arched chord in which the maximum stress is a constant will now be investigated.

Take an eight-panel truss, number the bottom panel points 0, 1, 2, 3, 4 to the center, the top apices b, c, d, e. Suppose the load to be ten *kps* at each panel, then $R = 35$.

Make *bl* normal to *bc*; *c*2 normal to *cd* and *d*3 normal to *dc*; then for a constant stress (S) in the arched chord the length *bl* *c*2, *d*3 may be determined at once from the following moment equation:

About 1 $Ra = S \times b1$
About 2 $2Ra = S \times c2 - 10a$
About 3 $3Ra = S \times d3 - (30a)$
Length $b1 = R\,a \div S$
Length $c2 = (70a - 10a)/S$
Length $d3 = (105c - 30a)/S$



If $S = 1.6\ R$ then $bl = .875a$; $c2 = 1.07a$ and $d3 = 1.34a$.

Thus the particular form of the truss conforming to an arbitrary stress magnitude in the chord members may be readily determined except S is assumed equal to or less than the reaction R when the conditons can no longer be complied with.

16. Pauli's Girder is a girder having uniform stress intensity in the arched chord with Pratt webbing. The tangent offsets from the center outward increase roughly as the square of the distance from the center stopping at the hip joint, and such determination of the heights having given the center depth and elevation of the hip approximate under these imposed conditions the most economic and graceful form.

17. Uniform Section of the Chords is conducive to simplicity of detail and economy. The preceding investigation demonstrates the impossibility of uniform stresses in both chords, hence that triangulation which results in the least variation of both is preferable. A somewhat greater height at the hip than uniformity of arched chord stresses requires will render the end post stresses somewhat less than the section at mid span, reduce the stresses in the end web members, give necessary height for a rigid portal frame and result in a more economical diagram.

The web members in the truss diagram just figured for uniform maximum stress in the arched chords lack practical uniformity of triangulation which is corrected in the diagrams of designs illustrated later.

As uniform cross sections simplify detailing so also does the arrangement of the arched chord segments in EQUAL LENGTHS thereby permitting the same top lateral triangulation to suffice for all panels.

The economic depth of parallel chord diagrams of like number of panels may best be taken for the first approximation and the strain area worked out graphically and apportioned for web and horizontal flange or shear and moment and the exact depth may be determined from these numerical strain areas.

Whether this depth should be followed or lessened depends on the width of the structure whether single or double track as the over turning tendency of wind and the increased cost of erection with increased height is to be considered without losing sight of the fact that the vertical stiffness decreases as the depth is decreased. A considerable reduction of depth such as 10 percent from that of the minimum strain area does not greatly increase the weight as indicated by the investigation in Chapter XIV, Art. 3.

A height exceeding about three times the out to out width of the truss should be avoided by spreading the trusses more than the minimum clearance requires for the long span.

18. Economic Sections of Compression Members: In the present day design of top chord compression members and the same is true to some extent in the design of posts, the influence of the old commercial pool on the price of beams and channels is still in evidence. Some thirty years ago the mills fitted for the turning out of beams and channels charged a differential for these shapes considerably higher than the base price of plates and angles with the result that the fabricating shops used beams and channels as little as possible and fabricated their sections from plates and angles thus establishing a custom, the influence of which is marked even at the present day when the combination prices have long been abandoned.

In Section IV, page 82, a photograph of the failure of built up sections of plates and angles tested by the Bureau of Standards is shown. The plates and angles which are riveted together and assumed in column theory to act as a solid are there split apart, the rivets sheared and separated as the ultimate strength is developed.

Because the mills are today rolling eye beam sections up to 30 inches in depth with a flange width of 15 inches and an area of section of 60 inches and upward, there is no necessity for the large amount of shop work involved in riveting plates and angles together for an eye or a channel section in the makeup of a chord and then assuming that the elements thus imperfectly fastened together act as a solid notwithstanding ample experimental evidence that they do not act in that manner when tested to destruction.

The simplest possible top chord is accordingly made up of two deep eye beam sections with a cover plate riveted to the top flange. Doo dab lacing with its pantograph action becomes unnecessary with a design of this kind. Cast steel separators flanged to be substantially riveted to the cover plate and top and bottom inside flanges of the eye beams and the webs render the three parts a unitary whole requiring only that the separator diaphragms be spaced apart not more than 40 L/R of the individual eye beams. These separators or cast diaphragm frames furnish resistance to torsional deformation, render the cover plate and the beam web stiffer under longitudinal compression, do not distort the section as lacing does and constitute a simple detail greatly reducing the shop work. Present day shop equipment renders cutting away one flange of the beam at the panel points for the proper attachment of the gussets a relatively inexpensive matter.

To cover the natural deficiencies of laminated plate and angle sections relatively short panel lengths have been adopted or truss sub-divisions which require the support of the compression members at one or more points in the panel length by sub-struts which function merely to support a member too weak to carry its stress between the main panel points and performs no other function except a detrimental one of increasing the secondary stresses in the frame and adding to its work of supporting dead weight.

Nick Nack triangulation may be economically avoided by the adoption of stable bulk dimensions for the compression members. This is particularly true in the posts. With the arch top chord four or eight angle sections laced with channels in the plane of bending and with angles in the transverse plane can be readily arranged with a radius of gyration sufficient so that no sub-supports are required in the height of the truss. The appearance of the frame is improved by the simplicity of the diagram and its economy enhanced in accordance with the fundamental principles of aesthetic design outlined in Art. 10, Chapter XXIII.

19. Eye Bar Bottom Chords lack the rigidity and directness of the transfer of stress from the web members to the chord that is possible in the makeup of the riveted frame. Their disadvantages in entailing dependence of the lateral system paneling upon that selected for the main truss have been commented upon in the preceding article and in summing this discussion there remains to be considered the amount of metal which is required for the details of the eye bar heads against the excess of the gross section over the net section in the makeup of the riveted member and in the splicing of the same at the panel points.

Now for 25-foot panels the excess metal in the eye bar heads amounts to substantially twenty to twenty-five percent of the total section. In making up a riveted section, 10 to 12 percent is enough to cover the metal cut out by necessary rivets. Hence there is ample advantage in the weight of the gross section of the riveted tension member over that of the eye bar to cover additional details of lacing and leave something to the good. Again, because with the riveted section a member 50, 60 or 80 feet can be designed so that its depth to length ratio is unobjectionable from the standpoint of rigidity, whereas even if we could make an attachment to the eye bars for the laterals, their depth ratio to panel length would render them too limber under the vibration of train loading.

Universal Mill plates can be secured in lengths from 70 to 100 feet, and in making up a heavy tension chord, the plates may be dovetailed at the panel points so that the gussets for connecting the web members and laterals will furnish the additional material necessary to satisfactorily splice these chords and we may therefore conclude that the riveted bottom chord member is more economical than the eye bar chord with the advantage of greater stiffness and better details for the lateral connections.

20. Riveted Construction is Preferable to the Pin Connected Structure for all railroad bridge spans because not only are the riveted chords cheaper than the eye bar chord as discussed in the preceding article but the floor beam connections are simplified.

Square ended beams may be used and cutting the web out for the pin and the expensive gussets to reinforce the web which would otherwise be too weak are avoided. For the diagonals members can be used longer than eye bars may be secured without a break and greater stiffness and economy results. Accordingly we may conclude that the riveted railroad bridge for standard E60 loading can always be built with less metal and greater stiffness may be secured with riveted sections than it is possible to design using eye bars and pin connections.

Continuity of stringers should be embodied in all designs to secure greater stiffness of the floor and the avoidance of tension on the rivet heads of the stringer connections and the over strain of the connection angles themselves, which has in the past constituted a most troublesome deficiency in the maintenance of old bridges with shallow stringers.

In securing this continuity, we may depend upon milling the end of the bottom chord angles and developing a larger part of their strength in this way by bearing against angles of the floor beams supplemented by the short cover plate crimped over the bottom of the flange. For the continuity of the top flange it is better to make the stringers deeper than the beam so that the flange angles may be spliced over the beam by an angle on each side and a cover plate on top. In this way a very rigid and substantial detail may be worked out.

Where clearance limits the depth of the stringer to a minimum continuity of the stringers is most advantageous in increasing the floor stiffness. Tho under these circumstances the above method of splicing the top flange would need modification and a different makeup.

22. Expansion Joints: The coefficient of expansion of iron and steel is .0000065 Fahr. Range of temperature to which the bridge structure is subjected varies from 100 to 140 degrees. For the latter range in 150 feet the expansion would amount to 1.17 inches. Multiplied by the modulus of elasticity of 30,000,000 divided by the length would represent a stress of 19,600 pounds per square inch. It is accordingly desirable to take care of this expansion by allowing the bearings of the truss to slide at one end upon the abutment. Usual provision is that of an allowance of about one inch extreme range per 100 feet.

It is interesting indeed to look backward and note that as iron was introduced in the construction of the first iron viaducts, proper provision for expansion was thought to render it essential that the bracing of the viaduct posts should be carried out in timber instead of metal. When Mr. L. S. Buffington, a Minneapolis architect, first proposed a 28-story building with a steel skeleton frame in the '80s, an interesting comment appeared in the Architectural News:

> "L. S. Buffington, an architect, claims to have invented a system of construction to build buildings of iron. He does not know that the expansion and contraction of iron would crack all the plaster, that in a few years there would only be the shell left. Iron is good in its place but not to build buildings entirely of."

New York Sun, 1888:

> "L. S. Buffington, an architect, is building a wonderful building on paper which is to be 28 stories high. To be sure this is probably the project of a crank, but the cranks of one generation are sometimes the prophets of the next."

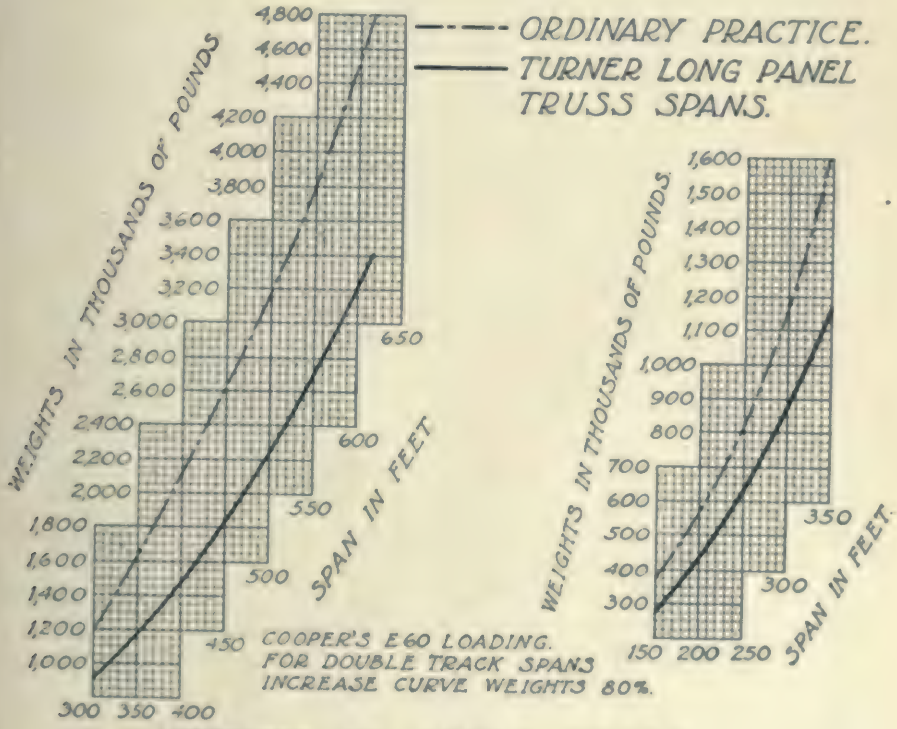
This fear of expansion so neatly illustrated in the example cited and in the architectural papers of a decade ago caused the introduction of multifarious joints in viaducts of the nature of joggle joints rather than expansion joints. This fear gave rise to the device of many complicated provisions for taking care of this slow moving force with ample push behind it. Why we should need complicated roller nests and bearings which fill with dust, rust so that they oppose a large resistance instead of properly machined surface scored in the usual manner of bearings for lubrication and an ordinary alemite gun connection to shoot in a pint of grease once in two or three years is a matter which with present day knowledge of machine design is difficult to explain. Somebody started to build roller nests fifty or sixty years ago and we have been doing it persistently ever since regardless of the additional expense, the general deficiency and lack of dust-proofness of the joint.

Because the truss deflects under load, a pin connection for the shoe is the most practical detail and the shoes may be more substantially and cheaply made of cast steel than by fabricating from plates and shapes. Castings from the electric steel furnace can be recommended because of the freedom of this grade of metal from blow holes, its uniform quality and reasonable cost.

22. In the Makeup of the Truss Design the First Step is to Determine the Truss Diagram. The lines of the truss should be simple, representing clearly the lines of strength. They should stand out boldly eliminating nick nack aggregations of triangles such as the K-bracing, Petit sub-divisions and the like which add to the secondary stresses, increase the weight, destroy the simplicity and mar the aesthetic appearance of the frame. Economic triangulation with but a limited departure from the equilateral should govern the makeup of the diagram. The same law holds true in the triangulation of the lateral system, but because the distance between the chords in terms of span length is small for the lateral truss, the main truss panel should be an even multiple of those of the lateral system.

In the floor system continuity of stringers increases the stiffness and reduces the weight. The number of pieces to handle in the shop may be greatly reduced or even cut in two over that of ordinary practice with an increase in stiffness of the finished structure, a reduction in the cost of fabrication and a saving of thirty percent of the weight The multiple panel division of the bottom laterals not only reduces the weight of the laterals themselves but tends to nearly double the lateral rigidity of the system under impact of train loading because of their more favorable triangulation.

These facts may be visualized and illustrated graphically by the curve of steel weights for different span lengths following the ordinary practice and following that here recommended for the Turner type of long panel bridges with multiple sub-divisions of the lateral system and continuous stringers. The curve of average weights of customary practice is reproduced from the pamphlet of Heath and Milligan, which runs a trifle under the weights of the Chicago, Milwaukee & St. Paul railroad as plotted in the Engineers Handbook by Ketcham.



COOPER'S E 60 LOADING.
FOR DOUBLE TRACK SPANS
INCREASE CURVE WEIGHTS 80%.

WEIGHTS OF SINGLE TRACK
THROUGH RIVETED SPANS

23. Summary: During the past thirty years, the equivalent weights per foot of engine and train loads have increased enormously. The engines have more than trebled in weight and whereas 3,000 pounds a foot was considered a heavy train load in the '90s, 6,000 pounds a foot is not uncommon on ore carrying trains tho it generally exceeds materially the freight train weights for mixed traffic. During this period of great increase in weight of loading, little change in the makeup of trusses and the relative dimensions of their triangulation is noticeable. In other words, designers have not kept pace by modification of the frame in keeping with its loading and in that respect their progress has been far short of other lines of engineering endeavor.

CHAPTER XXVI

DETAILS OF THE TRUSS SPAN

1. Even vs. Odd Panels: Referring to the Figure of the Minimum Strain Area, pp. 46–47, it is observed the curve for consecutive, even and odd panels is a zig zag line. That is while the points which represent consecutive even panel numbers lie in a smooth curve as in the case of the Warren with the sub-verticals, so also will the strain area coefficients for odd numbers of panels of either the Pratt or Warren type present a like uniform curve of coefficients of lower magnitudes than that for the even number of panels. The reason for this divergence is found in the fact that the moment magnitudes at each panel point when the bridge is fully loaded form a polygon lying upon a uniform curve approximately parabolic. On the odd panel numbers the middle ordinate of the polygonal figure is less than that of the uniform curve. Thus the difference between the maximum moment to which the three-panel truss is subjected when compared to a two-panel or a four-panel truss is 8/9ths, and this divergence in moment magnitude decreases as the number of panels into which the span is divided increases.

2. Variation in Strain Area Coefficients is caused not only by the divergence of the even and odd panels, but also by the departure of the configuration from the equilateral triangle. In this respect the Pratt truss of more than four panels has an advantage over the Warren truss because the web members of the Pratt which resist the greatest shears toward the ends are inclined at a more favorable angle. Thus the end shear in the end panel of an eight-panel Warren is brought upon two inclined members in the Warren, but on one only in the Pratt. And, notwithstanding the fact that the secant of the angle for the vertical is more favorable for the Warren, we find a divergence of between five and six percent in favor of the Pratt. With the eight-panel Warren with the sub-verticals, however, because the angle of inclination of the web members carrying the load toward the support is still more favorable, the strain area coefficient is greatly reduced and the economic height increased.

If, for example, we were to design a 200-foot Warren truss with sub-verticals taking the depth as that for the minimum strain area of approximately $L/4$ in order to keep the top chord panel lengths within about 50 feet, we should have eight panels and a strain area coefficient of 1.5 *WL*. On the other hand, the same depth reducing the number of panels to four in the long span type of the diagram we would find our strain area coefficients 1.25 for the long span type against 1.5 with the sub-vertical subdivisions of the plain Warren type, a difference of over 16 percent in the strain area coefficient of the truss, increased by the percentage of details in the increased number of joints, additional field riveting, increased number of floor beams from three to seven, and doubling the number of stringers.

Again, the section of the Warren with the sub-verticals would present a greater variation in the cross section of the top and bottom chord members than would be found in the camel-back in addition to the increased riveting and the hangers which merely transfer the load vertically without bringing it nearer the support.

If we compare the camel-back of four panels to the Warren truss of four panels, we get a coefficient of 1.25 against a strain area coefficient of 1.32 showing the material advantage of the camel-back of lesser and more practical depth over that of the Warren.

For the 150-foot span, for example, we might select six panels 25 feet and compare the strain area coefficient of the Pratt at 1.52 with that of the three-panel camel-back of 1.06, again indicating the striking economy of the long panel and inclined chord.

In this case, the three-panel Warren of .32 *L* in depth against the .25 *L* for practical depth of the camel-back shows a strain area but slightly increased, but a truss 48 feet high for 150-foot span gives undue length of web member and requiring for those which resist compression increased and uneconomical sections which are obviated in the shallower camel-back truss.

3. The Weight of the Floor System is to be investigated from the economic standpoint as a variable in keeping with the variation in the stringer moments, beam shears and moments and details involved.

Now the floor of the railroad bridge is made up of stringers and floor beams supporting the same load. Hence the weight in the floor beams will bear that ratio to the weight of the stringers approximately as the length of the beam is to the length of the stringer and a reduction in the weight of the one will mean that ratio of reduction in the total weight of floor as its weight, divided by the sum of the two parts.

In investigating the advantage of the long panel in respect to the floor beam, we may refer to the diagram in Chapter XXIII and note that the beam shears increase substantially as a straight line function from 25 to 70 feet, so that the sum of the flange sections of the beams will be substantially the same for an indefinite number of panels whether the panels were long or short, providing the floor beams were made the same depth. But with the increase in panel length the stringers are increased in depth and the floor beam may be accordingly increased and some economy effected in that way.

Again, in (N) panels of any truss the number of beams is $N-1$. Hence the total flange sections for a given depth are reduced in the proportion of $(N-1) \div N$. In other words, in the case of four panels and six panels compared the total beam weight with the beams of the same depth would be as 66 to 83, approximately. Taking into consideration the greater depth and reduced number of webs because of the fewer beams, the beam weight with three panels in 150 feet as against six would approximate 5 8ths the weight for the three-panel bridge that is required for the six-panel under live load without impact, but the percentage of impact with the live load for two long panels fully loaded or 100 feet is .6 whereas for 50 feet it is 85 percent, which would reduce the ratio of 5 8ths to approximately 1/2.

Consider the stringer: For a 20-foot panel the equivalent uniform load for the E60 is 12,500 pounds a foot; for a 40-foot panel is it 10,000 pounds a foot. The bending moment of the applied load were it the same in both cases, would be four times as great for the 40-foot stringer as for the 20-foot, but reducing it for the divergence in loads, the static momeht to be provided for would be 3.2 times as great for the longer panel that it is for the shorter.

Now the impact for the 40-foot stringer would be .9; for the 20-foot it would be 1.1 times the static live load effect, leaving an approximate ratio of 2.9 times as great effective moment for the stringer in the one case as in the other. But for the 40-foot stringer a depth of web nearly twice as great may be used without reducing the cross section of the flanges to dimensions too attenuated from the standpoint of thickness to properly support the track and resist compression.

Accordingly there would be approximately a twenty-five percent increase in the stringer weight due to the deeper web, a reduction of fifty percent in the weight of the floor beams with a resultant negligible difference in the total weight of the floor when the additional number of connections are taken into consideration.

As the length of the span and the number of panels increase there would be an increase in the weight per foot of the floor system by the reduction in the relative economy of the beam weights on the basis just figured, but this increase in the weight of the floor would be small in comparison to the large saving in the elimination of sub-divisions of the main truss proper and we may therefore conclude that in the most economic design of the railroad bridge truss we should center our attention upon the economic design of the triangulation of the truss proper without regard to the paneling of the floor system, except in case clearance conditions require an uneconomically shallow floor construction.

4. Extreme Length of Panel is limited by the cross sectional area of the compression chords and the economic bulk dimensions which the cross section permits the designer to use.

In Section IV, pages 13 and 14, it will be noted that for L/R less than 60, for the values of $n = 1/3$ or $n = 1/2$, the reduction in working stress for column action is small and as the compression chords of the bridge structure form a large unit of the make-up of the total weight, it is desirable from the economic standpoint that the slenderness ratio of these members be kept less than 60, preferably below 50.

In the make-up of the compression member, it is desirable that the bending stress be reduced by a liberal proportion of the metal in the flanges. It is further desirable to reduce as far as possible the shop work of riveting plates and angles together. Thus for the end posts and top chord of a three-panel 150-foot span, two 20-inch I-beams and a 30-inch cover plate may be used. By riveting the cover plate to both flanges at the top of the beams and connecting them by light cast steel diaphragms riveted both to the web, the inside flanges of the beams and to the cover plate at intervals of about 6½ to 7 feet, no lacing with its pantograph action will be required. The bottom chord of the truss design has been made up of two 18-inch channels laced and battened and the main ties of a single I-beam without shop work except punching in the flanges for connection at the ends.

By dividing the lateral system of the bottom chord into nine panels against three of the main truss, the triangulation of its webbing will not depart far from the equilateral triangle and the greatest rigidity and economy results. Moreover, by rigidly connecting the bottom flanges of the stringer to the laterals, the stringers, bottom chords and the laterals form a unitary horizontal truss of great rigidity operating because of the relatively large section of the stringers in such wise that the wind strains in the chord are reduced by the longitudinal stresses induced in the stringer thru its co-operation with the chord. And hence the lateral rigidity of the floor becomes approximately double that of the ordinary six-panel structure with identical lateral and main truss panels.

5. The Width of the End Post constitutes the depth of the portal legs, the flexural rigidity of which limits the sway of the top chords under the lateral forces of wind and impact. The distance between the webs in case of a box section determines the lateral bulk dimensions of the web compression members and whether their ratio of L/R may be the most economic, i. e., less than 50 or little above this slenderness ratio. With the camel-back, because the shears are in a large part resisted by the arched top chord, four large angles laced four sides make an excellent web member and when so used, the angle flanges should be turned in rather than out to reduce the twisting tendency of the leg to a minimum so that additional width outside the spacing of the chord webs may not be counted upon. In the 150-strain sheet illustrated the cover is 30 inches for two 20″ I's in the makeup of the end post and top chords in the 200-foot span, 34 inches by 3/8 inch cover plate is used and a 48 cover for a four hundred-foot span the chord and end post section of which is two 30″ I's and the 48 cover.

By utilizing the unitary rolled web and flange of the Bethlehem I-beam section, the inherent weakness of the riveted laminated sections commonly used is obviated and shop work simplified and reduced.

When the required cross section of the chord exceeds 225 square inches, the octagon section adopted in the 770 foot simple span is most economical.

6. End Floor Beams represent a waste of metal and unnecessary field riveting of stringer and brackets to extend the floor to the abutment parapet.

The motive in design of the end floor beam is to provide a lower portal brace so restraining the end post that the bending moment from lateral forces may be halved by the rigid connection of the beam. But the floor beam usually frames to the end post in as awkward a manner as that of Mr. Hedrick's design of the 160-foot span or Mr. Modjeski's N. P. 150-foot thru truss spans illustrated in Chapter XXV.

Placing the stringers on independent pedestals, the lower portal frame may be designed solely for the purpose it is intended to serve and its position so selected that the most direct and efficient connections may be made, to wit, in keeping with the stringer depth the frame may be moved forward to that position which permits a direct connection.

7. **End Panel Lengths** should be shorter than the intermediate and center panels by approximately eight percent, because the maximum moment of an end span in a series of continuous spans is greater than for an interior span. Part of this desirable reduction in length may be secured by placing the outer edges of the stringer pedestals in line with the main truss pedestals thereby shortening the span of the stringer to less than the end main truss panel and increasing the cantilever overhang toward the parapet.

By such variation in panel length, the maximum positive stringer moment may be kept down to six-tenths the moment in a simply supported span of the longer central stringer and a constant flange section maintained thruout.

8. **Stringer Bracing** may be effected by cross frames at or close to the connections of stiff lateral bracing to the bottom flanges of the stringers. With a 60-foot main panel divided into three lateral system panels, such frames would be spaced about 9-foot centers and no diagonal bracing would be needed.

9. **Double Track Thru Trusses** because of the greater width of floor may be made deeper than single track spans and approach more closely the theoretical depth for the minimum strain area.

10. **Quality of Metal** should be such as to possess the greatest strength consistent with toughness. In the discussion of Sec. IV, it has been shown that the grade of bridge steel recommended by the current manufacturer's handbooks is inferior for the compression member to the wrought iron of years ago and a stronger grade of metal is desirable. Quoting Jas. Christie, Trans. A. Soc. C. E., 1893:

> "Extensive evidence is on record indicating that steel of good quality of moderate thickness, not much exceeding 80,000 lbs. tensile strength is not any more, and frequently not as much injured as wrought iron by the process of punching and shearing."
>
> "The physical effect of punching and shearing as denoted by the tensile test, are
>
> "Reduction of ductility; elevation of yield point, reduction of ultimate tensile strength. In very thin material the disturbance is less than in thick, in fact a degree of thinness is reached where this disturbance practically ceases as the thickness is increased, the injury becomes more evident."
>
> "The best results with the least expenditure can be obtained by punching - - - - and reaming sufficiently to remove the material disturbed by punching."

Instead of metal of 55,000 to 65,000 ultimate tensile strength, 75,000 to 85,000 is to be preferred, which shall stand a cold positive bend test in specimens 5/8 inch thick of 180° and a reverse bend of the same amount without sign of fracture in either side of the bend about a radius of twice the thickness. With open hearth steel in which phosphorus and sulphur are kept below the usual tolerance, manganese below 30 and silicon between the limit of 25 and 40 points, there is no difficulty in securing steel which will show a minimum elongation of 22% in eight inches and a yield point value above 48,000 pounds per square inch if not finished at too high a temperature.

Such steel may be punched 1/8 inch less than the nominal diameter of the rivet for thicknesses under three-quarters and reamed to a sixteenth greater diameter with a twist drill, all holes being rymered or burred. For greater thicknesses the metal should be drilled. A basic unit working stress of 18,000 pounds per square inch may be figured on with a higher degree of safety than the 15,000 pounds of the A. R. E. A. specification for the softer metal which it provides for. The quality of metal and workmanship will be covered more completely in the specification presented in the last chapter of this section.

11. Paint is effective as a preservative coating only as it is applied to dry metal cleaned of scale and rust. Parts inaccessible after assembly should be given two good coats before assembly and riveting. The number of paint formulas are legion. The requirements of a good shop coat are first that it will spread smoothly and without too much labor, i. e., that it be not too thick, that it dry quickly and possess a tough body as a base for the field coat. Pure boiled linseed oil does not fill the latter requirement. With Red Lead it is difficult to get an evenly applied coat. The following formula for the shop coat will give satisfactory results if properly applied to *clean dry metal:*

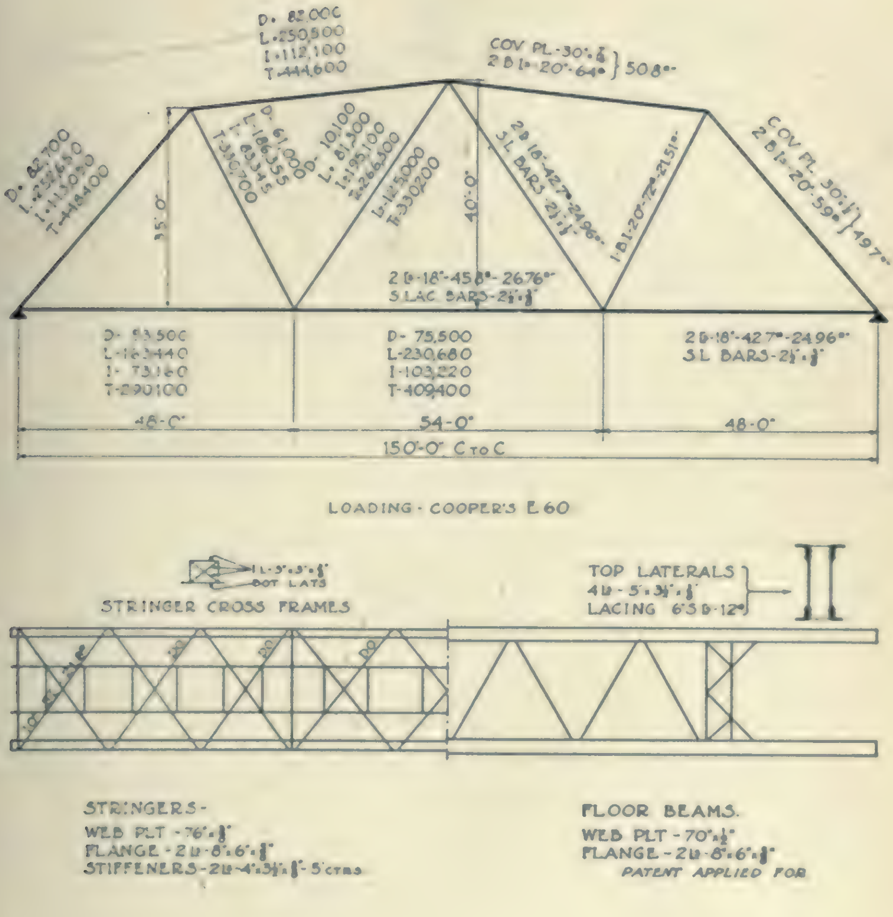
5 lbs. sublimed blue lead
3 lbs. red lead
3 ounces lamp black
2 quarts boiled linseed oil.

For the field coat the writer regards it probable that aluminum paint will be largely used in the future.

The ordinary pigments, red lead, blue lead, zinc oxide, iron oxide, etc., are composed of particles granular in form though exceedingly small. Aluminum powder is essentially flake like in character because of the stamping process of its manufacture.

When the polished powder is mixed in a varnish vehicle, the flakes come to the surface quickly forming a continuous film of aluminum in layer upon layer much like fish scales. The particles appear to float on the liquid, but it is apparently a surface tension effect which holds them there. The phenomena is descriptively termed "leafing." It confers an attractive appearance almost as highly reflecting as a sheet of polished aluminum. It is opaque to light and hence protects the vehicle underneath from deterioration by the action of sunlight. Only by the slow weathering of the upper layers of aluminum and vehicle are successive lower layers exposed. Its protective power is due not only to its opacity to light but also to its high reflectivity. Because it reflects so much and absorbs so little of the radiant energy of the rays which fall upon it, the temperature of the paint and the material underneath remain substantially lower than with paints of lower reflectivity. While the cost of the powder per pound is high it requires but two pounds to a gallon of paint and its spreading capacity is such that its cost in use compares favorably with the better grades of paint on the market.

12. Summary: In the foregoing discussion an effort has been made to show the weight of divergent elements entering into the economic problem of the railroad truss design without following the tedious method of figuring different designs and comparing the computed weights. Comparison by computed weights fails to convey a concise conception of the different elements entering into the design. Even and odd panel trusses compared in this way tho designed and figured for the same depth, the question whether the depth be not too great for the even panel or vice versa is left in doubt. We might figure various arrangements of beams and stringers in the floor and arrive at a generalized result, but such computation is tedious and may be obviated by the more direct investigation of the applied forces and their distribution as has been done in the preceding chapter. General designs and strain sheets of several fairly economic camel-back thru truss spans have been worked out to illustrate the author's type of bridge frame and the comparison of the estimated weights are shown in the diagram representing the steel weights of truss spans of the common short panel Petit or Parker type of truss and the long panel type here developed from which the relative economy is apparent.


D- 82,700
L-252,650
I-113,050
T-448,400
D- 82,000
L-250,500
I-112,100
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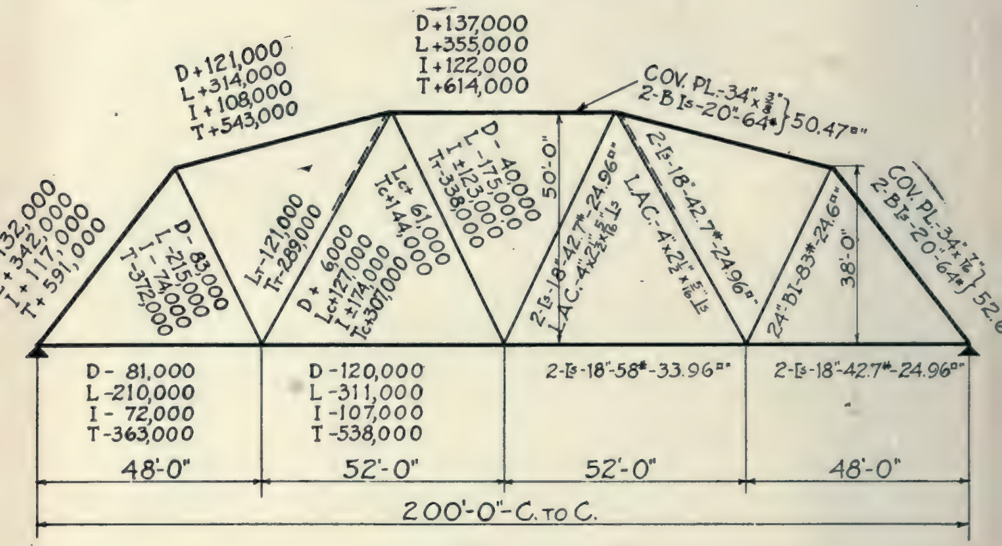
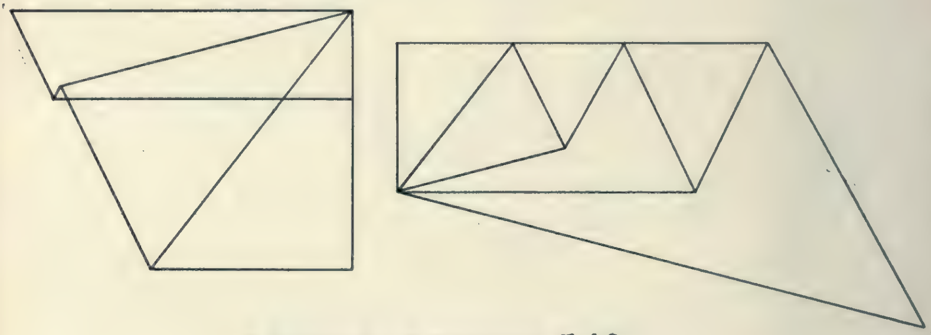
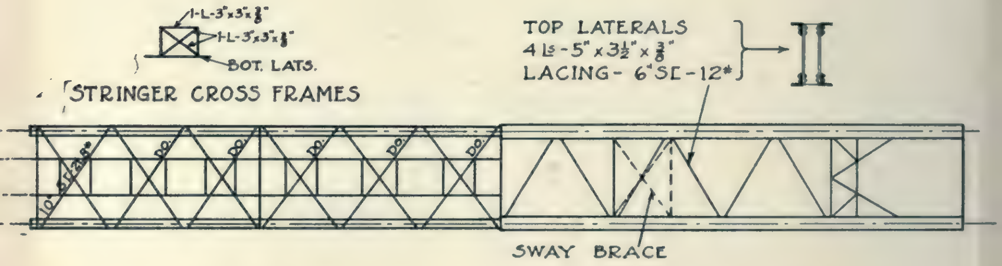
D- 61,000
L-186,355
I- 83,345
T-330,700
D- 10,100
L- 81,500
I-195,100
T-266,500
L-125,000
T-330,200
COV PL-30"×⅝
2 B I-20"-64#
508□"
35'-0"
40'-0"
2 B 18"-42.7#-24.96□"
SL BARS-2½"×⅜"
I-B I-20"-72#-21.51□"
COV PL-30"×½"
2 B I-20"-59#
497□"
2 B-18"-45.8#-26.76□"
SLAC BARS-2½"×⅜"
D- 53,500
L-163,440
I- 73,160
T-290,100
D- 75,500
L-230,680
I-103,220
T-409,400
2 B-18"-42.7#-24.96□"
SL BARS-2½"×⅜"
48'-0"
54'-0"
48'-0"
150'-0" C TO C
LOADING - COOPER'S E 60
1 L-5"×3"×⅜"
BOT LATS
STRINGER CROSS FRAMES
TOP LATERALS
4 L-5"×3½"×⅜"
LACING 6"-5 B-12#
STRINGERS-
WEB PLT -76"×⅜"
FLANGE -2 L-8"×6"×⅜"
STIFFENERS-2 L-4"×3½"×⅜"-5' CTRS
FLOOR BEAMS.
WEB PLT -70"×½"
FLANGE -2 L-8"×6"×⅜"
PATENT APPLIED FOR





LOADING- COOPER'S E60.



STRINGERS.
WEB PLT. - 76"x$\frac{3}{8}$"
FLANGE - 2 Ls-8"x6"x$\frac{5}{8}$"
STIFFENERS - 2 Ls-4"x3$\frac{1}{2}$"x$\frac{3}{8}$"-5' CTRS.

FLOOR BEAMS.
WEB PLT. -70"x$\frac{1}{2}$"
FLANGE - 2 Ls-8"x6"x$\frac{5}{8}$"

PATENT APPLIED FOR.

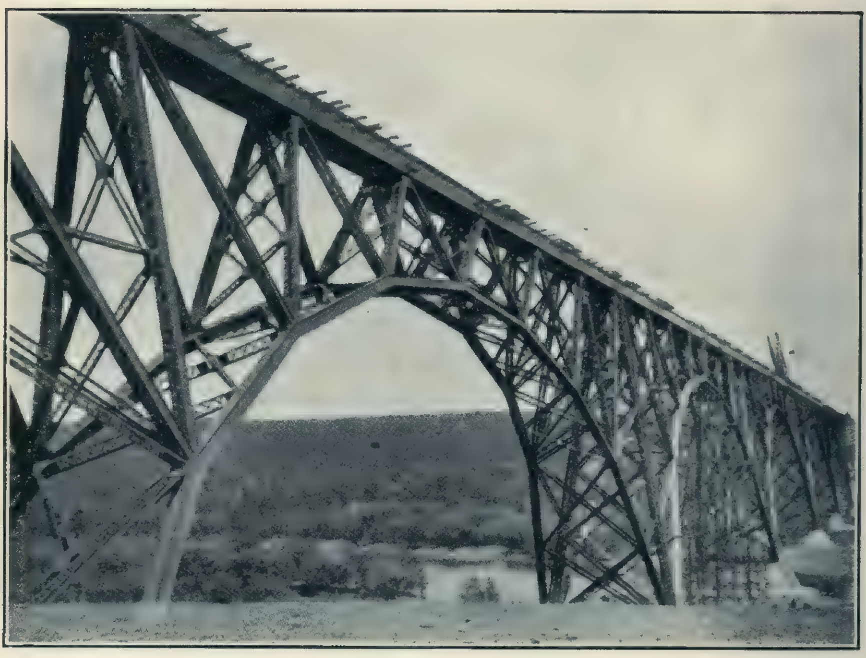
CHAPTER XXVII

ECONOMIC DESIGN OF THE VIADUCT

1. The Division of the Crossing into its Elementary Spans depends on the character of the foundation strata, flood conditions or navigation clearances which determine the relative cost of the sub-structure and the super-structure. Because the floor system of the thru bridge weighs approximately the same per lineal foot regardless of the length of span, it may be treated as a constant and eliminated from the consideration of those variables which may be modified to reduce the cost to a minimum. Eliminating this constant in case thru spans are required, the cost of the trusses erected with their lateral bracing should equal the cost of the foundation for the minimum total cost. On the other hand, because the cost of the super-structure is a determinate element, while the cost of the sub-structure may involve more expense than expected and cannot be figured with the same degree of certainty it is conservative to see that the cost of the super-structure exceeds the sub-structure rather than that the reverse relation holds true.

2. The Deck Bridge is to be Preferred to the Thru Bridge where the clearance or contour of the crossing permits, because in the deck structure a combination floor system and top chord is practical and economical, whereas in the thru bridge the combination of the bottom chord and floor system is generally impracticable or lacking in economy.

The accompanying cut represents the Soo Line crossing over the St. Croix, consisting of five 350-foot arch spans and these 180 feet from the deck to the river with plate girder viaduct approaches. Each of these spans was designed with 50-foot top chord panel lengths and the combination box top chord and floor stringer. The floor system in this case consisted of two box girders, two 60 by 3/8 webs, cover plate 34 by ½, four top angles 6 by 4 by ½, four bottom angles 6 by 4 by 7/8, spaced 9 feet centers, giving a steel floor substantially 12 feet wide out to out. The inclined main trusses render the structure one of the stiffest railroad bridges in the country.



The adoption of the 350-foot span was dictated by the reduced cost of the foundations over that for a plate girder viaduct of the usual type of construction with the additional advantage of reduced danger of scour around the pedestals as in this particular location the river bed consisted of silt and fine sand for a depth of about 120 feet to sand rock.

A plate girder trestle viaduct cost less money for superstructure than the arch, but the foundations far more than offset the saving. The longer spans were thus more economical and their safety enhanced by the reduced number and larger size of the pile supported pedestals.

3. The Plate Girder Viaduct or the Deck Lattice Girder and Trestle Structure which may be erected by using an upper traveler for both setting the towers and placing the girders, is most economical for heights up to 150 feet, when foundation or flood conditions permit. Braced tower bents in a long viaduct of this character should be placed about every fourth span and have a longitudinal length in the neighborhood of about 60 feet for a 150-foot trestle and 40 feet for a trestle half that height. The three intermediate spans may be advantageously made continuous for both stiffness and economy and the single columns be given a breadth so that their ratio of L/R should not exceed 50 to 60.

Friction joints should be used for the sliding plates so that the weight of the train clamps them and the expansion joint is removed from the class of joggle joints commented upon in the preceding chapter.

Stiff rigid bracing should be employed in the trestle bents. Horizontal members are sometimes added to a double intersection, Warren bracing, but this addition is objectionable for the reason that the tie forces the Warren bracing to resist a unit compression from vertical loading equal nearly one-half that of the unit compression in the vertical posts. When it is serving thus to transmit vertical load to the pier, it cannot be counted upon for its full efficiency as longitudinal bracing which was the motive or intent in providing it. Lack of simplicity, economy and neatness of appearance results from the insertion of these redundant and unscientific members in the bracing of the trestle bent.

4. Simple Spans vs. Continuous Bridges. With parallel chords, the cross section and the weight of the chord decrease as the depth of the frame increases, and, conversely, the weight of the web increases as the depth increases. Therefore, in such a frame, the economic weight approaches a minimum when the weight of the web equals the weight of the chords, and this relation holds true whether the frame is continuous or simple.

Economic Depth Greater for Simple Span: Continuity renders the live load shears resisted by the webbing somewhat greater than those in the simple span. Therefore, the web of the continuous span would be heavier than that for the simple span, but the maximum moments are smaller for the continuous span. If it is a through span with inclined end posts, the chord length is shorter in the simple span than in the continuous span, as the chords for economy should equal the weight of the web, then, because of this difference in the webbing, the economic depth of the simple span would be greater than the economic depth of the continuous span. Parallel chord bridges, however, are not economical for moderately long spans and, for trusses of variable depth, there is a more radical difference in the relative economic depth than for the case of parallel chord bridges of the two respective types.

Economic Depth:—Non-Parallel Chords—Simple Spans. The reverse law of summation of shears and moments is most favorable to the simple span. Thus, shear increases from mid span to the support, and the applied moment increases from the support to mid span. Accordingly, the simple span truss may be made deep at the center and of small depth at the end. With the greater depth at the center where the shear is the least, the web members, although long, are light. With decreased depth toward the support, the heavy web members are short, so that by this arrangement the weight of the web becomes greatly reduced over the case of parallel chords. Moreover, because of its inclination, the inclined chord functions in a dual capacity, resisting both shear and moment, still further reducing the weight of the web which, with the parallel chords, was one-half the total truss weight.

Contrast this reverse law of summation, so helpful to the economy of the simple span, with the conditions which are unfavorable in the continuous span. The moment is greatest numerically over the support where the shear is greatest and to obtain depth to resist this moment, heavy web members become long in place of the correspondingly short members of the simple span resisting the same shear. The economic depth for the simple span thus becomes from twenty to thirty percent greater than for the continuous truss.

As, with economic height, the web of the simple span would reduce to about one-third the weight of the continuous type, and the chord lengths would be less, the conclusion appears inevitable that instead of the economy claimed for the continuous frame, it is generally lacking in economy to the extent of twenty to thirty percent, as compared with the simple truss frame for moderate spans.

5. Stiffness: It is generally assumed that a continuous truss bridge is stiffer than a simple span when each are economically designed. The origin of this mistaken idea may be traced to the fact that in a rolled beam or plate girder, the section of the web which resists horizontal shear distortion is several times as great as the cross section of the flange which resists bending the deflection due to shear is almost negligible and the deflections depend substantially on the moment resisting areas.

In the truss, on the contrary, the maximum unit stresses in the web members are substantially the same as the maximum unit stresses in the chord members and that portion of the deflection contributed by the web in the truss economically designed nearly equals or balances that contributed by the chords.

Now the deflections caused by shear deformation sum from the end of the span toward the center and with the simple truss span with the chord arching downward the shear strain area of the web members decreases because their length is decreased.

Now the end web members with an arched top chord in the simple span will be about half as long as those of the continuous span and the deflection due to the web strain will be reduced by more than half. Again, if the depth at mid span is one-third greater the moment deflections will exceed but little those of the continuous frame when an interior span is fully loaded and the adjacent spans unloaded and thus the advantage of stiffness of the continuous truss bridge over the simple truss may be reversed by this divergence of economic depths. This difference in favor of the simple span is however insufficient to offset material economy did that in fact pertain to the continuous bridge type.

Experience with the continuous draw spans gives the practical engineer a better basis for comparison of the stiffness of two span continuous structures with simple span structures of the dimensions of the length of the arm than the relations of moment forces presented in works on bridge design. The results of such observation are diametrically opposed to the conclusions customarily drawn from incomplete theory of resistance based on moment while neglecting shear.

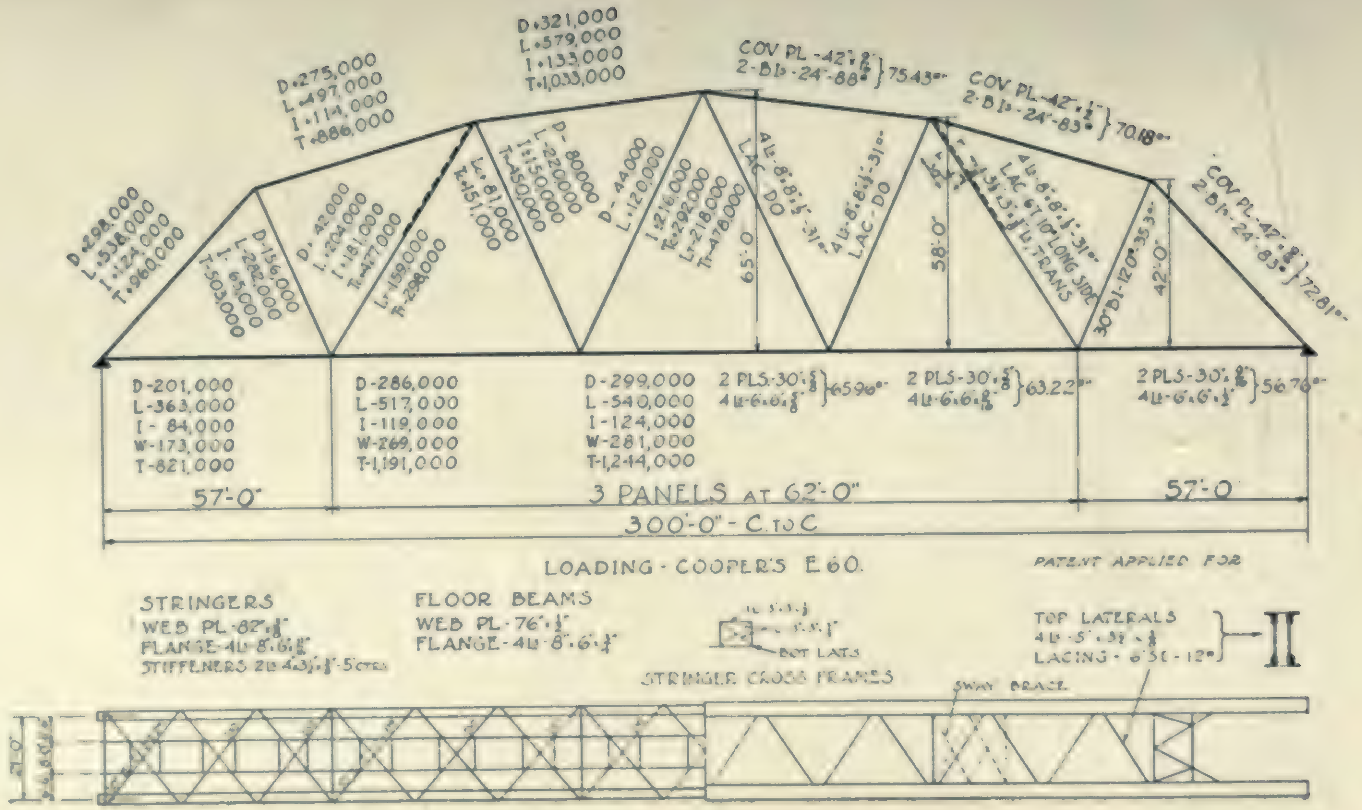
6. Diverse Economic Law of Chord Inclination in Simple and Continuous Spans: As the apportionment of the efficiency of inclination of the inclined chord in the simple span is as the cosine of its angle of inclination in resisting horizontal deformation of moment, and the sine of its angle of inclination in resisting the vertical deformation of shear, its efficiency in resisting shear and moment is as the secant of its angle of inclination is to unity, compared with horizontal chords on a unit weight basis.

Because the bending moment in the cantilever portion of the continuous span is of the reverse sign, there is a reversal in the law of the economic efficiency of the inclined chord member. For negative moment its efficiency in resisting horizontal moment stresses is as the cosine of its angle of inclination just as it is in the arched chord of the simple span, but its efficiency in resisting vertical shear tho likewise proportional to its sign of inclination is negative instead of positive for it carries the load vertically a distance upward from the point of reaction thereby increasing instead of decreasing the material necessary to support the shear as is the case with the inclined chord where the moment is positive.

As the inclination of the member over the support increases its lever arm, a reduction in the section required results so that the economies of inclination are the algebraic sum of the plus and minus advantages noted instead of the consecutive positive summation as with the inclined chords of the simple truss span.

As an exemplification of the theory just outlined, a cut of the Sciotoville bridge is reproduced and the writer's design of a 770-foot double track, simple span with an arched top chord, two of which are approximately but two-thirds the weight of the continuous truss of two spans. Part of this divergence in weight is attributable to more economic panel lengths, the elimination of nominal members (some 450 tons) which reduce the unsupported length of main compression members so lacking in bulk that they are unable to support themselves between the truss panel points and a further difference lies in the consistent application of the principle of continuity to floor stringers where economy can be obtained by continuity instead of disregarding this principle as has inconsistently been done by advocates of the continuous framed truss. The author is a believer in the efficacy of continuity, but would apply it consistently only where advantage and greater stiffness can be attained by its application as is the case with plate girder or rolled beam stringers and shallow deck truss spans with parallel chords.

7. Minimum Strain Areas of Continuous and Simple Spans Compared: In the Theory and Calculation of Continuous Bridges, No. 25 of the Van Nostrand series, Mansfield Merriman compares the third span of a seven-span continuous Warren girder with a simple span on the basis of uniform load and the strains in members of identical triangulation and lengths. On page 27, he sums the strains of the web members in the simple and continuous truss and finds them equal, while the sum of the strains in the chords are more than twice as great in the simple truss as in the continuous truss and from this he argues that there is a saving in material of 37½ percent by using a continuous truss instead of a simple truss. But reducing these stresses to strain areas, we find that the depth selected for the continuous truss approaches the economic depth very closely and the magnitude of the strain area coefficient is 1.07, while the parabolic arch chord simple span for economic depth would give a strain area coefficient of 86 percent of that of the continuous truss span. Since the strain area coefficient of the economic diagrams for the respective types are comparable in respect to the weight required for a given loading instead of 37 percent in favor of the continuous span, it would appear there is 16


D.298,000
L.538,000
I.124,000
T.960,000
D.275,000
L.497,000
I.114,000
T.886,000
D.321,000
L.579,000
I.133,000
T.1,033,000
D-156,000
L-282,000
I. 65,000
T-503,000
D. 42,000
L.204,000
I.181,000
T.427,000
L.159,000
T.298,000
L.81,000
T.151,000
D- 80000
L-220000
I.150,000
T.450,000
D- 44000
L.120,000
I.216,000
T.292,000
L.218,000
T.478,000
COV PL-42"×⅝"
2-B1s-24"-88#
75.43#
COV PL-42"×½"
2-B1s-24"-83#
70.18#
COV PL-42"×⅝"
2-B1s-24"-83#
72.81#
4Ls-8"×8"×⅝"-31#
LAC DO
4Ls-8"×8"×⅝"-31#
LAC DO
4Ls-6"×8"×⅝"-31#
LAC 6T 10" LONG SIDE
3½"×3"×⅝" Ls TRANS
30" B1-120#-35.3#
65'-0
58'-0"
42'-0"
D-201,000
L-363,000
I- 84,000
W-173,000
T-821,000
D-286,000
L-517,000
I-119,000
W-269,000
T-1,191,000
D-299,000
L-540,000
I-124,000
W-281,000
T-1,244,000
2 PLS-30"×⅝"
4Ls-6"×6"×⅝"
65.96#
2 PLS-30"×⅝"
4Ls-6"×6"×⅝"
63.22#
2 PLS-30"×⅝"
4Ls-6"×6"×½"
56.76#
57'-0"
3 PANELS AT 62'-0"
57'-0
300'-0"-C. to C
LOADING-COOPER'S E60.
PATENT APPLIED FOR
STRINGERS
WEB PL-82"×⅜"
FLANGE-4Ls-8"×6"×⅝"
STIFFENERS 2Ls 4"×3½"×⅜"-5 CTRS
FLOOR BEAMS
WEB PL-76"×½"
FLANGE-4Ls-8"×6"×⅝"
BOT LATS
STRINGER CROSS FRAMES
SWAY BRACE
TOP LATERALS
4Ls-5"×3½"×⅜"
LACING-6"[-12#
17'-0"

percent in favor of the simple span and certainly we may not look for improvement in relative economy in the continuous bridge for live load when the positive moments are a maximum when the span itself is loaded and the adjacent spans unloaded and the magnitude of the shears are affected by the diverse loaded or unloaded conditions of other spans than that of the particular one under consideration. Moreover, because of greater economic depth, the arched chord simple span is stiffer than the best continuous span that may be devised.

If our theoretical conclusions are to be reasonably accurate from the practical standpoint, the premise on which they are calculated must be based on complete theory rather than partial theory which considers moment while neglecting shear; or which erroneously assumes that economic height of the continuous and simple trusses may be assumed alike for purposes of comparison instead of determining economic height for each individual type and comparing the properties on such an unbiased basis.

Computation of the weight of simple spans, two of which are comparable to the largest example of continuous bridge construction in America at Sciotoville, emphasizes this point and it should be noted that in this suggested design the details are liberal, that the lacing bars range from 8″ H beams down to 7″ 14-3/4 pound channels and 6-inch ship channels; that the gusset plates at the joints are about 11 feet wide by 12 to 15 feet long and 5/8 inches thick; that the angles which make up the posts are strengthened against twisting by cast steel separator blocks leaving only those short lengths unstayed which Mr. Christie has shown by his extensive experiments to develop much greater ultimate strength by block action than the yield point value of the section.

In reducing the sum of the stresses to a basis of equivalent length, Prof. Merriman in his discussion has followed a line of reasoning somewhat similar to that of our strain area coefficients but in considering only a uniform load he accords to the web members of the continuous bridge an equal advantage to that enjoyed by the simply supported structure, whereas if we consider in turn his Fig. 10, page 90, and his computation of the shears opposite page 99 for another and different continuous girder, we at once see that the strain area of the web of the continuous truss for the same depth and triangulation may readily run twenty-five percent greater than that of the simply supported girder and hence 13 percent of the 37 percent advantage disappears.

Again, because the economic depth of the parallel chord con-

tinuous bridge is less than the economic depth of the parallel chord simply supported span, this reduced advantage is again decreased so that it is possible under the most favorable conditions to materialize a total economy of 15 or 16 percent with the continuous truss as compared with the simply supported truss with parallel chords having a like triangulation and the same number of panels.

Where the height is limited as in deck bridges, so that the depth of the simply supported span is less than its economic depth, the comparison becomes more favorable for the continuous structure. Its percentage of advantage decreases with the decrease in number of spans and even the advantage just figured cannot be materialized with the continuity of a two-span structure. This small percentage of advantage with parallel chords disappears with arched top chords with which the advantage rests with the simply supported span.

Summarizing, we may profit by continuity of the stringers in bridge floors. Occasionally we may profit by continuity in deck truss spans, but the construction of thru spans on the principle of the continuous truss leads to economic waste and more material than is required for simple trusses properly designed.

8. Cantilever vs. Simple Span: As the cantilever is in general a three-span continuous truss hinged at the points of inflection of the center span, the same comparison with simple spans that has been presented above is applicable. Its adoption has generally been dictated by ease of erection which in many cases can be provided for with simple truss frames with but nominal additional equipment provided the trusses have stiff bottom chords capable of resisting compression during erection. As such riveted types of construction are more economical than the pin type, there are many locations where three simple spans of long length may be as readily erected by providing a toggle back stay to the hip of the center span and cantilevering from the hip of the fixed spans. Material saving in dead weight may thus be effected and greater stiffness of the super-structure secured.

9. Minimum Strain Area Diagrams Applied to Determine the Form of Truss and Economic Panel Lengths will now be Considered. In pages 46 and 47 minimum strain area diagrams for Pratt, Warren and Warren sub-vertical trusses are computed and the form and depth shown graphically with their strain area coefficients. Depths are shown in percentages of L and the strain area in coefficients of WL.

Examining these curves, the increase in the strain area of the Warren sub-vertical between four and six panels is proportional to the percentages of W which functions in producing the stress and strain in the truss, but for a larger number of panels the increase in the strain area coefficient is determined more largely by the departure of the triangulation of the shear carrying members from the *equi-lateral triangle.* In addition to the strain areas for these elementary simple types, the curve of strain areas for the camel back with the Warren webbing is shown.

It will be noted that for a 200-foot span of the camel back type with four panels, the top chords are supported at less than fifty-foot intervals, while if we were to compare it with the four-panel Warren with sub-verticals, the top chord would be 100 feet in length without support which is an impractical relation of length to dimension of the compression chord. Hence, we will be forced, if we compare it with the Warren sub-vertical, to take an eight-panel in which the strain area is 1.50 against a strain area of approximately 1.23 for the camel back. Again, for the ten-panel camel back of 770 feet, a reasonable top chord length and a practical height of 135 feet are selected, while the ten-panel Warren sub-vertical would give 170-feet height and 134-feet chord length which would be impractical from the standpoint of erection and top heaviness of the structure and makeup of compression members.

Thus the subdivision of the truss must be made with reference to practical limitations of height which increase the cost of erection and may render the truss type top heavy under the lateral forces of wind. For that reason in the longer span bridge simple Warren web systems with arched top chords are the most economical.

In them there is no transfer of the load back and forth thru the depth of the truss without carrying it toward the support. There are no members which do not function in truss action by the elimination of sub-panel divisions.

In interpreting the minimum strain area diagram and its indications of economic depth, the relative percentage of details required by web members and the chords is to be given consideration. The lacing and latticing of the relatively light sections of the web members renders their percentage of details nearly double that of the chord members and this increase in weight for satisfactory details tends to reduce the economic depth of the minimum strain area from five to ten percent dependent on the type and makeup of the truss. For the long span camel back the minimum strain area involves a greater height than permissible for the economic depth by ten to fifteen percent, because of erection difficulties, top heaviness and this divergence in percentage of details in the chords and web members. In the long span, the higher the truss the more expensive the traveler and the higher the erection cost becomes. The variation in weight for a considerable departure from the exact economic depth is small. Its increase is approximately a parabolic function; at first slow—then more and more rapid as the square of the divergence from the proportions of economic depth as may be appreciated from the computations of the rolled beam and girders of the previous chapter.

10. Arch and Cantilever Arch Trusses: Where the foundation conditions are favorable and the height sufficient, the arch is more economical than the simple truss because the thrust displaces the bottom chord and thus eliminates an element of considerable weight from the design. Its elimination permits a larger economic depth for the minimum strain area diagram than that for any form of simple truss.

A modification of the arch type is presented by the cantilever arch which may be utilized in certain locations for long span construction. It possesses an advantage over the cantilever in that its effective depth may be made much greater in proportion to the span length than consistent for continuous or cantilever construction.

Further consideration will be given to these forms in the discussion of long span bridges.

The suspension bridge is sometimes classed as an inverted arch, and may be designed of chains of eye bars or with cables made up of small wires spun from anchorage to anchorage, afterward leveled to the proper height and wrapped or bound together into main supporting cables.

The suspension type was used in the early days of bridge building for railroad traffic. The old suspension bridge over the Niagara gorge is one of the early examples of such constructions now replaced by the rigid two-hinged arch. The Niagara bridge had a span of 825 feet with two decks, the lower one carrying a highway 15 feet wide partially enclosed at the side by the timber stiffening trusses. The upper deck 24 feet wide, 245 feet above water, had a single track in the center and was floored over separating it from the floor below. The floors were suspended at intervals of 5 feet from the upper and lower cables which were four in number, 10-1/4 inches diameter, containing 520 wires in each. The masonry towers were 60 feet above the road. The bridge was braced laterally against wind by 56 wire guy ropes 1-1/4 inches diameter fastened to rocks below. It was completed in 1855 at a cost of $400,000. In 1897, the whole bridge was replaced by the present steel arch with a span of 550 feet.

In service, railroad suspension bridges have not been satisfactory in the matter of stiffness. That at Niagara, Frankfort, Vienna and Durham over the Tees have been removed and replaced. In 1857, a suspension bridge was started by Mr. Roebling to carry the Cincinnati & Southern over the Kentucky river at an elevation of 275 feet above water. The span of the cables was 1,230 feet, but after the towers were completed, building was discontinued owing to failure of the company. Twenty-two years later, in 1867, a continuous three-span structure designed by C. Shalee Smith was built across the gorge with a deck of 275 feet above the water, each span 375 feet in length on metal towers. After completion the chords were cut 75 feet shoreward from each tower making the end spans 300 feet and a center double cantilever girder 525 feet long. The girders were 37′3″ deep and 18′ centers. The trusses were replaced in 1910 by heavier ones 70 feet deep for a double track on wider towers and foundations.

A further discussion of the suspension bridges will be presented in a later chapter dealing with long span bridges.

CHAPTER XXVIII

FOUNDATIONS

1. As the economic cost of the viaduct depends jointly upon the cost of the sub- and super-structure a brief discussion of the fitness of different types of sub-structure under different conditions may be helpful to the student.

The natural material on which the structure rests is called its FOUNDATION BED. The design of a foundation may be defined as the exercise of engineering judgment in providing a safe and permanent base for the support of the super-structure.

The load on the foundation bed should be kept within the safe compressive resistance of the material of which it is composed under extreme conditions of loading and the worst conditions of storm or flood. To meet these requirements, a thorough exploration of the site and tests of the physical characteristics of the material of which the foundation bed consists is the first step.

As the bridge may be in any part of the world, so the material encountered in the foundation bed may vary as widely in character, as the diverse geological formations of the earth's surface which for our purposes may be roughly divided into rock and earth composed of various materials differing from each other in chemical composition and physical properties.

Rock forming minerals are largely comprised in two groups, the silica minerals and the calcarious minerals. Pure silica in its crystalline form is known as QUARTZ. Because of its hardness it better resists abrasion and decomposition than other minerals with which it is found associated, hence it forms the principal constituent of sandy deposits. Flint and agate are non-crystalline forms of silica. The cementing material in some sandstones and many other rocks is silica deposited from solution in a manner similar to the building up of stalagmite and stalactite deposits which all are familiar with.

Combinations of silica with various bases are second only to quartz in wide distribution. Silicate of alumina with soda lime and potash is a large constituent of granite and other igneous rocks. Granite is hard and durable in general when there is no iron in its makeup. Then it is readily decomposed.

By the decomposition of granite, silicate of aluminum, the base of clay, is formed, the mica and quartz remaining unchanged.

Chlorite, talc and soapstone are hydrated silicates lacking in hardness with a soapy texture. Sliding on the foundation bed or between parts of it with this as a base is to be guarded against.

Horn blende and augite are silicates of lime, iron and magnesium and alumina of frequent occurrence.

The carbonate of lime $CaCo_3$ when pure and crystallized is known as Iceland spar or calcite. It is soluble in water containing CO_2. Calcite in varying degrees of purity forms the base of limestone and marbles. The carbonate of lime and magnesia with generally some silica forms the Dolomitic Limestones and marbles.

According to the manner in which they have been formed, rocks are classified as igneous, sedimentary and metamorphic.

IGNEOUS rocks are those which have solidified from a molten condition. They may be of granular crystalline on glass like texture but without stratification. Granite, bassalt trap, lava, obsidian and pumice are examples.

SEDIMENTARY rocks were deposited from water born material such as clay, sand, marl, etc., the consolidation being effected under water by pressure and chemical action. As the materials were deposited in layers of different grades, stratifications are found in accord with the manner or rate of sedimentation.

After hardening, the original beds have frequently been subjected to upheaval and the inclination of the stratification should be considered with reference to sliding or disintegration in the foundation bed.

Strata of sand cemented into a rock body by silica, oxide of iron, or carbonate of lime, constitute the different varieties of sandstone, the durability of which is determined by the cementing material. Carbonate of lime being soluble in water containing carbonic acid, sandstone cemented by it weathers more readily and is not as reliable as though the cementing materials were silica or iron oxide.

Chalk is a soft limestone of earthy texture, white, buff or gray in color, chiefly composed of minute shells of formanifera.

Limestone is likewise often composed of calcium deposits of marine organisms. In some deposits fossils, shells or coral formation indicate its origin, whereas other limestones present no indication of organic origin. The presence of clay and other impurities often renders it difficult to distinguish by their appearance certain limestones from shale or some kinds of sandstones. The purer the limestone the more soluble it is and the more frequent the occurrence of caverns and voids in the ledge.

In the formation of oolitic limestones, thin layers of clay are found in the stratifications which disintegrate with freezing and thawing and prevent the rock from giving satisfaction as a building stone tho below the frost line it is satisfactory as a foundation bed.

Argillacious rocks contain clay with fine sand, mud, etc., and while some shales are hard and compact others deteriorate as exposed to the frost and weather. The shale clay which underlies a large area below the silt of the Missouri River bed in North Dakota forms a good foundation bed below water though it disintegrates when exposed to sun and frost at the surface.

The general effect of METAMORPHISM is to produce a hard durable rock. By heat from a nearby intrusion of molten rock, limestone is changed to a crystalline marble.

Quartzite, a crystalline rock of great hardness, is a metamorphosed sandstone, the change in character being effected either by heat or the chemical precipitation of silica.

Slate is a strong dense rock formed from clay shales. Gneiss is a laminated metamorphosed rock, some varieties of which resemble granite but the laminated aspect is characteristic.

Mica schists are liable to decomposition into small grains of mica and fine material which when wet acts as a quicksand.

EARTHY MATERIAL resulting from the disintegration of rock masses is broadly classed as EARTH. By the combined action of water and high temperature rock of such great strength as a feldspathic granite may disintegrate, the feldspar changing chemically into the hydrated silicate of aluminum known as clay, while the crystals of quartz mica or hornblende remain unchanged but become detached particles which are classed as sand.

By the mechanical action of frost, fragments are detached from the ledge of which they were a part and they become shifted about by various agencies.

As flowing water more readily transports small particles than large fragments, large pieces are moved intermittently during storm or flood and are deposited as soon as the current velocity slackens, the smaller particles being held in suspension longer are deposited in the order of their size, the larger first. Swift mountain streams constantly move and grind together the materials of their rocky beds forming the particles of sand and gravel which are washed down and deposited as the current slackens, first as beds of gravel, then sand bars, and finally in the slower moving lower levels as beds of silt and alluvium.

The action of glaciers is similar to that of the stream. The deposits differ from that of the stream in that with like material the classification as to size of the deposit is lacking. Boulders may occur in the surface of the glacial drift or may be disseminated as by chance thru the deposit.

The occurrence of glacial boulders thus gives little indication of the character of surrounding deposit. In river deposits on the contrary, the occurrence of boulders indicates that the bed of which they are a part has been thoroughly consolidated by current action which formed it and also because such deposits generally extend to compact material capable of resisting the action of rapidly flowing water. Sand, gravel or boulder strata deposited by wave action are not necessarily uniform in the gradation of material to any great depth and tests should be made to determine the nature of the material underlying the beach formations. When tributary streams discharge into the wide basin of a main river, they often deposit bars of gravel and boulders over lying silt clay or marl or other untrustworthy material deposited by the main stream and the investigation of such sites should be thorough.

2. Clay when pure is the product of decomposition of feldspar and consists of hydrated silica of aluminum. As encountered in the foundation bed, various impurities are mixed with the clay so that in general ordinary clay is a mixture of hydrated silicate of aluminum with very finely divided minerals. Mixtures of clay and sand are found varying from beds of nearly pure clay to that of nearly pure sand, rendering definite classification difficult. As found in excavations, clay is generally in a moist plastic condition, the amount of moisture varying greatly. On drying, clay shrinks in volume, loses its plasticity and becomes a firm coherent mass resembling sun dried brick. Loss of volume in drying out is considerable and depends on the amount of the impurities present, ranging from two or three percent to over fifteen with purer clays.

Compact moist clay is impervious to water in the sense that it cannot pass thru as it would thru course sand, but eventually it may become saturated and softened by water to a considerable depth. This is particularly true where there are seams in the clay or strata of sand thru which the water can percolate or penetrate. Such seams give rise to sliding and render the material dangerous where the foundation bed is on sloping ground.

In the author's first experience with blue clay of Winnipeg, he discovered that an excavation ten feet deep might cause almost horizontal motion of the material to start from a point seventy feet away. Excavation of a twelve-foot square area for the footings two-feet deep would slide nearly full under a shower of short duration. Notwithstanding that, under finished structures the same clay is regularly supporting more than four tons per square foot without excessive settlement. The reddish clay in the same district containing very little sand is exceedingly unstable when wet, the tower of a very artistic church built of beautiful cut Tyndale limestone collapsed within a month after its completion because of the instability of wet red clay in the foundation bed which appeared stable until saturated with water.

Any foundation bed in which change in the moisture content of the clay may be subject to wide variation is liable to undesirable settlement and it is well to examine the clay both in the wet and dry condition in forming final judgment as to its stability under the most unfavorable circumstances. The investigation of the deportment of structures supported by it if there are any in the vicinity is more to the point than mere loading tests on a small area. In general it may be said that the supporting power of the clay depends in a large measure on the amount of sand or other impurities which it may contain.

Ordinary stiff clay should not be loaded with more than two tons. If soft and plastic, two tons may produce inadmissible settlement. Clay containing so large a percentage of sand as to lose much of its plasticity may be loaded with four tons per square foot and stand up well. In bridge locations where the banks rise abruptly and rest upon clay strata, the site should be investigated for faults or evidence of slipping or sliding of the bank toward the river channel. The east abutment of the Northern Pacific Bridge over the Missouri at Bismarck was pushed toward the river several feet by such sliding presenting a difficult and expensive undertaking to rectify and maintain, and like difficulties are best avoided in locating the crossing where faults are not to be found.

3. **Hard Pan** is an earth through which it is very hard to dig, such as gravel and boulders cemented by clay or indurated clay. Sand and gravel partly cemented by a small amount of carbonate of lime or by iron-oxide is also classed as hard pan. Hard pan may be regarded as a sedimentary rock in process of formation. The processes of solidification be they physical or chemical may be found in all stages from the softest clay thru *hard pan* and shale to the hardest slate.

Cemented hard pan containing gravel has been frequently loaded more than ten tons per square foot and this may not be excessive, providing the hard pan is continuous to a hard substrata, a matter to be thoroughly investigated by borings for any important structure.

4. **The Safe Load on Hard Rock** may exceed the crushing strength of concrete or stone masonry, while any material worthy of the name of rock below frost line is capable of supporting from from 15 to 20 tons per square foot, providing the ledge is thick. Shell rock of limestone shale and slate underlaid by soft material possess less supporting power than the ledge and only exploration by boring will determine the facts.

5. **Strata of Gravel Boulders and Coarse Sand** protected from scour may safely carry five to six tons per square foot for surface pressure.

6. **Fine Sand,** if confined laterally is nearly as incompressible as COARSE sand, but the necessary lateral restraint is not present in the foundation bed and when surcharged with water, fine sand is so unstable that it is characterized as Quick Sand. Under the weight of a railroad embankment where a layer of three to four feet of quick sand underlies ten feet of clay, the quick sand flows laterally, the embankment settles vertically and the soil on each side rises in a wave or roll as it is forced upward by the hydrostatic pressure below. Mud and all semi-fluid strata present like characteristics.

7. **For the Foundations of Culverts and Retaining Walls** it will be usually a sufficient precaution after having dug and leveled off the foundation pits below the frost line and below the probability of washing out, to test the earth by a sounding rod or better by boring with a common wood auger 1½ inches diameter with pipe fittings. This will bring up samples and show the nature of the underlying strata and the depths that it continues firm and in that respect satisfactory. If there is from 15 to 20 feet of firm earth there will rarely be risk in proceeding with the concreting or masonry work because this thickness will in general be safe for an ordinary abutment even tho deeper it be underlaid by softer strata.

The bearing power of coarse sand, gravel and hard pan may be taken as five tons per square foot; blue clay three tons and ordinary clay and silt not over two tons, while some soft clays cannot be trusted to support more than a ton and had better be piled or filled with a bed of rock and gravel.

8. **Exploration by Wash Borings** with the ordinary well

drilling rig can be made to a depth of 100 to 120 feet and samples of the material brought up at an expense of sixty to a hundred dollars per hole, and from such investigation fairly reliable data may be secured for the proper design of pier foundations. When an unreliable strata overlies a firm one, but at such a depth that excavation becomes unduly expensive and there is water to contend with, piles furnish the best manner of support for the foundations. Timber is permanent as long as it is kept wet, hence the top of the pile should be kept below the permanent water line.

9. The Bearing Power of the Pile (P) is usually figured when the piles are driven by a drop hammer by the formula

$$P = \frac{2\,W\,h}{S + 1}$$

For a pile driven with a steam hammer,

$$P = \frac{2\,W\,h}{S + 0.1}$$

In which P is the safe load in pounds, W the weight of the hammer in pounds, h the fall of the hammer in feet, and S penetration or sinking in inches under the last blow, assumed to be at an approximately uniform rate. These formulas were deduced for wood piles; but are the best there are for concrete piles.

Morsch gives the Brix formula as usually employed on the Continent:

$$p = \frac{h\,Q^2\,g}{2e\,(Q + g)^2}$$

wherein h is the fall of the hammer; Q the weight of the hammer; g the weight of the pile; e the penetration of the pile under the last blow; p is double the safe allowable load for the pile. The quantity e will naturally be the average of the last few blows.

Merriman gives the following comparison of actual loads upon piles and those computed by the formula—

Comparison of Actual and Computed Safe Loads of Piles

Length of Pile, feet	Weight Hammer, W, lbs.	Height of Fall, h, feet	Last Penetration, s, in.	Load, lbs.		Character of Soil	Locality
				Observed	Computed		
53	2000	4.0	8.5	13,333	1,690	Almost fluid mud....	Aquia Creek, Va.
25	1600	25.0	3.0	22,400	19,750	Soft muddy bottom..	East St. Louis, Ill.
35	1700	25.0	2.0	44,800	28,300	Mud, 30 feet deep...	Perth Amboy, N. J.
35	910	5.0	0.35	62,500	6,740	Mud, sand, and clay.	Proctorsville, La.
....	1900	29.0	1.5	75,000	44,100	Stiff clay...........	Buffalo, N. Y.
91	2300	22.0	1.75	75,000	36,800	Mud 60 ft., fine sand 6 ft..............	Annapolis, Md.
73	2300	33.5	3.75	34,000	32,500	Water 12 ft., mud 61 ft................	Annapolis, Md.
30	2300	22.0	2.0	38,000	33,700	Sand................	Annapolis, Md.
33	2300	22.0	1.0	110,000	50,600	Sand...............	Annapolis, Md.

From this comparison it is observed that the formula is fairly accurate only for relatively firm strata. In quick sand and semi-viscous mud or in silt, below the permanent water line the material settles about the pile so materially increasing its frictional resistance that a pile whose penetration under the last blow may be an inch may not budge after settling a day or two until loosened up under three or four blows of the same ram falling the same height. Under such conditions, the formula is not an inerrant guide. The spacing of the piles in alluvial earth has a marked effect on their bearing capacity. When driven too close together, driving one pile will force upwards its neighbor so that about three feet center to center is a minimum for the best results in soil of this character. When, however, the piles are driven thru clay to hard pan or firm strata, they may be spaced as closely as two feet six inches centers, i. e., when the pile resistance is merely frictional, the minimum spacing is greater than when it is predominantly that of bearing toward or near the point. Unusually high bearing pressure reported for piles in alluvial delta soil are accounted for by the cap bearing and the combined resistance of the distribution of the weight from the area of the cap and the friction of the piles effecting a spreading of the load laterally thru the alluvial earth.

For bridge piers in streams in which the bottom is clay and gravel which does not scour badly and extends to a considerable depth, piles may be driven at about 3 feet centers until penetration does not exceed one-fourth or one-eighth inch under the last blow of the ram, care being taken not to broom the pile by over driving. Piles are then cut off close above the low water level. A box is placed around the pier to prevent the flow of water and a concrete not leaner than a 1-2-4 mix is deposited about the piles with a tremmy. Care must be exercised that no separation of the concrete takes place by dropping it thru the water, but rather that it be deposited under water without dropping thru it any material distance. The end of the tremmy is to be kept just above the level of the concrete as it is deposited. In this way concrete was deposited at depths of over 70 feet below water in the Detroit tunnel and cores taken from this concrete tested from 7,000 to 8,000 pounds per square inch compressive strength. This method of building the pier saves expensive coffer dams and if the work is carefully executed it is satisfactory in locations where wash may be prevented around the base of the pier by a mattress of brush held in place by heavy rip rap.

10. Pile Driving Practice. A thorough exploration of the site by borings and by driving test piles is a prerequisite of economic design to determine the diameter, length and method of driving for the service expected.

The specifications governing the penetration of a pile should be adapted to the character of earth which the pile is to penetrate. It is more important that a suitable length be put in place without injury than that its penetration should be a specific distance under a given blow or series of blows. Avoid overdriving which is indicated by bending and kicking of the pile and bouncing of the hammer.

The steam hammer is more effective than the drop hammer in securing penetration without injury to the pile. The weight of the steam hammer for heavy piles should run from six to eight tons.

When the earth consists of compact coarse sand and gravel, the use of a jet and steel shoes for the pile may be advantageous.

When piles must be driven thru muck to a hard stratum at the foot, they may be treated as columns and braced for trestle construction, but are unsuitable for supporting masonry piers.

Where the bearing capacity of a pile depends predominantly upon skin friction unless a record of bearing capacity is available at the same site, it should be verified by loading not earlier than two days after driving in order that it may develop its full resistance by settling of the material about it. Pile foundations become unsuitable for the support of Bridge masonry where overburden above rock is too shallow to permit the pile to develop satisfactory lateral resistance. Under such conditions the masonry should be carried thru the overlying material to the hard strata below.

Where considerable excavation becomes necessary to preserve the wood piles by keeping them below the permanent water line, it may be cheaper to sink a hollow concrete shell to the hard underlying strata and fill it with cyclopean concrete.

11. Timber Piles for Heavy Foundations should be cut from sound timber free from worm holes, decay, ring shakes, unsound and loose knots or defects which impair strength. Oak, fir, tamarack, maple, spruce, redwood or cypress may be used.

Piles should be cut from sound live timber when the sap is down. They should be butt cut above the ground swell and have a uniform taper from butt to point. A line drawn from the center of the butt to the center of the tip should lie wholly within the body of the pile.

All knots to be trimmed close to the body of the pile. The minimum dimension at the tip should be 9 inches and a maximum at the butt of 20 inches.

12. The Old Method of Constructing Bridge Piers was after driving the piles to cut them off below water as low as possible, capping them with a grillage of timber and then build the masonry pier thereon. Simple bending resistance for the pile only was thus opposed to down stream motion and some cases have been observed where the shift of a railroad bridge pier was eight or ten inches down stream. The improved method is that of cutting the piles not more than two feet above extreme low water as they will keep wet by capillary attraction to this height if sealed with concrete on all sides. The pile embedded in the concrete in this way is restrained at the top so that the bending moment upon it is halved and its stiffness quadrupled over that of the pile which bears only upon the grillage at its top end.

Wood piles are less expensive than concrete piles, but for permanence they cannot extend above the low water level except the small amount the seal in the concrete permits. Accordingly, where the base of the pier masonry is above the permanent water line and the strata is soft, concrete piles may be advantageously used though more expensive than timber.

13. Piers Carried Down to Hard Material are the most satisfactory type of construction. If the depth does not exceed 15 to 18 feet, the area occupied by the pier may be cofferdammed with sheet piling properly braced, the excavation made and the pier built up in the open. For depths greater than this in general it is better to make the pier its own cofferdam providing it with a steel cutting edge and sinking it by dredging from the interior, following methods similar but not identical to those employed in putting down the deepest foundations such as that at Hawkesbury and Poughkeepsie Bridges.

In bridging the Missouri River between Bismarck and Mandan, the author designed the concrete piers to be sunk in this way and four were successfully sunk to the shale clay or hard strata at depths varying from 60 to 90 feet below water without the use of air. Where the material is shifting silt for fifty or sixty feet as in this section of the bed of the Missouri River and shale clay is encountered below this level, the cutting edges satisfactorily seal the pier shell after reaching the clay so that it can be pumped out and the excavation thru the hard material carried down to the desired depth in the open. Where, on the other hand, the pier shell is sunk to a satisfactory bed of a hard but pervious material at sufficient depth to escape danger of scour, a thick seal of concrete may best be placed by tremmy and the shell then pumped out and filled in the dry with cyclopean concrete.

In sinking the concrete shell base has many advantages over timber cribs for its bearing may be made a true cutting edge against the blunt timber edges and stone filled weighting pockets of the timber crib.

While the borings and general character of the stream may indicate an improbability of encountering undue difficulty from sunken logs, heavy boulders and other obstacles to excavation, the sinking determines the actual facts.

Reinforcement of the hollow pier shells in such wise that pneumatic pressure may be applied, is a desirable precaution increasing the strength and costing but a moderate amount for insurance against unforseen contingencies.

14. The Dimensions of the Bridge Pier must be such as to withstand the impact of drift at flood periods and that of ice floes and jams, logs, etc., where such occur. The striking energy of a moving body increases as the mass and the square of its velocity. The velocity of the current which causes the drift is swift at mid stream while back water and currents are caused by the greater friction near the banks than in the deeper water render the striking energy of drift on a pier near shore either negligible or a minute fraction of that to which the channel piers are exposed. Braced cylindrical piers of reinforced concrete may serve well for piers near shore, while the river piers in large streams carrying a heavy run of ice in the spring must have mass and be given a broad base.

The footings of the river piers in the Missouri River crossing at

Bismarck were given a base of 60 by 23 feet, the lower section being 14 feet 6 inches deep with vertical sides, followed by a shaft 16 feet wide with three dredging pockets. Above water the pier was narrowed down to 12 feet and fitted with a substantial ice breaking nose. The shore piers consisted of a pair of concrete cylinders twenty feet each in diameter at the base, stepped in to 14 feet, ten feet above the cutting edge with a nine-foot dredging core. A tie beam nine feet thick by twelve feet high connected them at water level, fitted with a vertical up stream cutting edge.

The bases on shale clay were figured approximately for five tons per square foot pressure *deducting from the total superimposed weight the displacement of the pier at 100 pounds per cubic foot.*

Steering or controlling the pier shell in sinking may be best effected by making the sides of the base vertical of a height from half the width of the base when the shell is cylindrical to five-eighths when rectangular. This limitation of the height of the vertical surface to the step back or reduced widths of the shaft greatly reduces the skin friction, opposing sinking and gives ample height for flaring back the cutting edge on its inner face and rigidly joining the reinforced concrete base and shaft shells.

As the excavation by clam shell dredge may be carried on through water as readily as in the dry, no pumping expense is incurred until the excavation has reached a strata the clam is unable to claw out. If the strata penetrated be considered insufficient to resist sliding or lacking in stability at the depth attained, the material may be loosened up by the judicious use of dynamite and removed by the clam as it is broken up. If the strata be impervious as in clay, the pier may be pumped out and the excavation proceed in the dry chopping and loosening the material in the open. If the bottom be pervious and the excavation very troublesome, resort may be had to the freezing process for sealing the base to facilitate open excavation.

In general, the method above outlined is the least expensive, most rapid and satisfactory method of sinking foundations, sometimes costing less than piling. In streams that have been used for logging, open dredging and removal of logs should precede the placing of the pier shell.

Winter construction of piers is most favorable when the ice forms with sufficient thickness and can be counted upon for two to two and a half months. The base may then be built on top of the ice with sufficient height of shaft located in its proper position and lowered into position by cutting thru the ice. The concrete needs to be not leaner than a one-two-four mix, placed hot and its hardening accelerated by steam. Forty-eight hours is sufficient time for a twenty-foot section to harden sufficiently so that dredging may be proceeded with.

15. The Compressed Air or pneumatic process has been used in sinking many bridge foundations to moderate depths. A base or caisson generally of timber with cutting edges similar to the base described for open dredge work is constructed and made practically air tight. After being placed in position, the water is forced out of the working chamber between the cutting edges and the work is carried on as tho in a great diving bell.

The maximum depth to which the compressed air process has been applied has been 113 feet on caisson No. 4 of the Memphis Bridge, and the next deepest was the East abutment of the Eads Bridge in St. Louis. The working time in compressed air for depths less than 40 feet may be eight hours per day with a visit to the open air for lunch, but when the pressure becomes greater, the working time must be shortened. At the Eads bridge at a depth of 100 to 115 feet, the men were able to work only two shifts of two hours each and even then they remained only a part of the time in the working chamber. At Memphis, between 80 and 90 feet, the men worked two shifts of two hours each and below 90 feet, three shifts of one hour each.

Thus the compressed air process is very expensive for the lower depths. Special provision must be made for the care of the men, a hospital and medical attention provided. The process is much less rapid than the open dredge method altho the latter, where numerous boulders and sunken logs are encountered, may not be as readily dealt with.

An air lock may be readily attached to the top of a pier shell, sunk by open dredging or the freezing process may be applied to seal the bottom and carry on an excavation in the open. But occasions are rare which warrant such procedure.

16. Masonry: Thirty years ago railroad engineers were somewhat opposed to the use of concrete piers and abutments. As the permanence of concrete when the cement and aggregate are what they should be is becoming more and more generally recognized, this prejudice is rapidly disappearing. With the exception of granite, quartzite and the hardest building stones only, concrete is the most permanent material that bridge piers may be built of, providing the sand used is clean, coarse quartz sand free from shale and the broken stone is a hard material such as trap, granite, quartzite or other stone which is known to weather well without disintegration under the action of frost. With the introduction of aluminate cement which acquires double the strength and greater hardness than ordinary Portland cement, the concrete pier may be expected to withstand the abrasion of ice and drift as well as tho it were faced with granite, in locations where it is subjected to the most severe service.

The concrete pier has the advantage that the shells can be rapidly constructed, moulded to any shape and the interior may be filled with cyclopean masonry at a very low cost. In filling the shells, alternate skips of rock and concrete may be dropped fifty or sixty feet in place and as long as the stones are clean they will bed themselves thoroughly in the plastic matrix.

17. Summary: In the preceding discussion it has been the intent to present a general survey only of the foundation problem covering the author's views of some phases of difficult foundations which are customarily executed in a more expensive manner than warranted by the conditions encountered rather than to enter into a detailed description of methods upon which volumes might readily be written.

CHAPTER XXIX

LONG SPAN RAILROAD BRIDGE STRUCTURES

1. Erection constitutes the major problem in building any unusally long span. The width of floor, weight to be carried, foundation and clearance conditions determine the rational type of structure and the arrangement of its parts. Thus, if the loads are light and the width of the roadway is relatively narrow as in highway structures, the suspension type is the only choice, but with wide roadway for the combined railroad and highway bridge the arch, cantilever arch or cantilever frame are preferable from the standpoint of rigidity and economy. For either of the latter types cantilever erection of the main trusses is necessary. The cantilever has within itself a frame work adapted to this method of erection while the arch has not. But the cantilever is harder to connect than the two halves of an arch built out as a cantilever. This advantage in the erection of the cantilever can best be overcome in an arch only by designing the floor system to function in a dual manner, first as back stays for cantilever erection of the main trusses and finally as floor members. If the skew back of the arch is not too far above supporting rock, less masonry is required in the abutments than for the support and anchorage of the cantilever.

In the arch but one force of the external couple of the applied loads is resisted by the steel while the other is resisted by the horizontal thrust of the skew back. In the cantilever, on the contrary, both forces must be resisted by steel sections. The practical net saving that may be realized would involve consideration of the amount of excess erection material required other than involved in the cantilever.

2. The Competition for the Sydney Harbor Bridge in Australia furnishes an interesting study of various types of structure as devised by English, French, Canadian and American engineers. The minimum span requirements for this structure were that of 1600 ft. for the cantilever and 1,650 ft. for the arch; total length including approaches for the cantilever 3,810 ft. and a total length including the approaches of 3,770 ft. for the arch structure. The specifications under which the competition was conducted provided:—

> 43. "If economy and enhanced appearance can be obtained by so doing, the Plans exhibited with and forming a part of this Specification may be varied by Tenderers in so far as

> the length of spans, the outline of trusses, the details of bracing, the number and pitch of cross girders and stringers, the details of decking and handrailing, the masonry-faced piers and abutments and the detail arrangements generally are concerned. The salient features, however, must be observed; the distance between centers of main piers cannot be decreased, although the length of the suspended span and Harbour arms of cantilevers may be varied, provided the distance of 1,600 feet center to center of main piers and the distance of 1,650 feet centers of bearings for the main arch span are observed. Similarly the length of Shore arms of cantilevers and approach spans may be varied, but the total overall length of Bridge, viz., 3,810 feet for the Cantilever Bridge, or 3,770 for the Arch Bridge, must be tendered for. Tenderers must submit their reasons for any variations proposed, and all work so varied must be equal in quality to that shown on Plans and specified.
>
> "The make-up of the various members and the details of construction are left to the judgment of Tenderer, but these must conform in every respect with the requirements of this Specification. All material must be of the class, quality, and strength specified. All workmanship both in superstructure and substructure must be the very best in accordance with the latest modern practice, and the Bridge and every part thereof must be capable of carrying the loads specified, based on the units tresses set out in Specification."

The deck was to provide for four railway tracks loaded by coupled electric locomotives weighing 180 tons followed by a train load of 2,200 pounds per foot on each track. A paved roadway 57 ft. wide between curbs and two 10-ft. foot walks. The main system was to provide for a live load of 12,000 pounds per lineal foot. The deck system of the roadway and sidewalks to provide for 100 pounds per square foot of loading and the beams, joists, etc., to be designed for a motor lorry having a wheel base of 12½ ft., width of 8 ft., weight on the forward axle 9 tons, rear axle 18 tons. Impact 50 percent for railway stringers, for floor beams, railway loading 40 percent; roadway loading 20 percent; other members of the deck system 25 percent; impact for members of the main system carrying two panels 25 percent and inner girders of approach spans 15 percent; all other members 10 percent.

Tensile and compressive stresses on different grades of metal were specified as follows:

"Tension on Net Section	**lb. per sq. inch**
Eyebars, carbon steel	20,000
Eyebars, carbon steel, heat treated	45,000
Riveted members, carbon steel	18,000
Riveted members, silicon steel	23,500

Compression on Gross Section

Members with 1/r under 50, carbon steel. .14,000
Members with 1/r 50 and over, carbon steel.18,000-80 1/r
Members with 1/r under 50, silicon steel. 17,500
Members with 1/r 50 and over, silicon steel. 22,500-100 1/r

"If alloy steel, either nickel steel or chrome nickel steel is used, the primary unit stresses given above for carbon steel may be increased 45 percent for the tensile stresses, and 40 percent for the compressive bearing, bending and shearing stresses.

"If alloy steel, chrome nickel molybdenum steel is used, having the chemical composition and conforming with the physical tests specified in Clause 96 (a) the unit tensile stresses for carbon steel may be increased 75 percent, and the unit compressive stresses for carbon steel may be increased 60 percent, subject to the Tenderer satisfying the Chief Engineer that the physical tests specified for chrome nickel molybdenum steel, can be obtained from specimens taken from material as rolled into plates and shapes.

"When secondary stresses are included in any combination of stresses, as stated in Clause 136, paragraph 3, the primary stresses specified above may be increased 30 percent."

Various arch cantilever arch and braced suspension types were submitted. The weight or tonnage of each design was greater or less dependent on the grade of metal figured upon, whether carbon steel, silicon steel or nickel steel.

3. The K-Truss Cantilever Design submitted by Sir William Arrol & Company had an estimated weight of 57,650 long tons while a *cantilever design using similar K-bracking by the Canadian Bridge Company was tabulated as 38,060 tons. The working stresses adopted for the K-system by Arrol & Company were those allowed under the specification of nickel steel while the Canadian Bridge Company tendered on heat treated carbon steel bars with working stresses of 45,000 pounds per square inch for the tension members and otherwise used silicon steel largely in the main system and the suspended part of the main span. In the design of William Arrol & Company all tension members were figured for a working stress of approximately 26,100 pounds per square inch against a 45,000 pound working stress in the eye bar tension members of the Canadian design, a difference which would roughly account for the three-tenths reduction in the weight to reduce the two designs to a like basis the balance to be accounted for by shorter cantilever and shore arms of the Canadian design with its longer central span and difference in make-up of the floor system.

*See page 17.

A three-hinged arch submitted by Arrol & Company following closely the outline of the accepted design, figured on the basis of nickel steel stresses, had an estimated tonnage of 40,230 tons or 70 percent of the total tonnage of the cantilever design using the same grade of steel.

Now in the make-up of the design the comparison as far as steel weights are concerned should be on the basis of the arch span plus as much of the approach as would correspond with shore arm of the cantilever and on such a basis from Arrol & Company's figures we find the arch weight *two-thirds* as great as the cantilever weight.

4. Relative Stiffness of the Cantilever Designs of the K-System by the William Arrol Company and the Canadian Bridge Company may be approximated directly from the divergence in working stresses. Since the tension members of the Arrol design were figured for a net working stress of 26,000, those of the Canadian Bridge for 45,000 and because the gross section would be greater than the net in the riveted section the deflection of at least fifty percent greater with the Canadian design than with the Arrol design of K-system cantilever frame should be expected.

The Government report states that the computed deflection due to live load was 25 inches for the Canadian design and 16½ inches for the Arrol design against a live load deflection of 6¾ inches for the three-hinged arch.

5. The Relative Weights of the Arch and Cantilever Arch Designs of the successful competitors, Dorman Long & Company, are tabulated at substantially 50,000 English tons for the arch design against 56,800 for the cantilever arch, or roughly the weight of the arch is 85 percent of the cantilever arch comparing the weight of metal in the approach length from out to out of the cantilever arch and including with the arch sufficient approach span metal to cover the same total length. As these two designs for the same grade of metal were figured by the same firm the comparison is more direct than that afforded by the other designs. Their cantilever design weighed 65,400 tons against 56,800 for their cantilever arch.

6. Rigidity and Distortion are treated by Chief Engineer J. J. C. Bradfield in his official report, as follows:

> "All structures under the action of the rolling load deform as the load passes from one end of the bridge to the other. The relative amount of deflection under live load at the center of the span is a criterion of the rigidity of the structure, and its suitability for the heavy railway traffic.

Dealing first with bridges of the suspension type, the deflection under live load of the braced chain suspension bridge designed by Mr. Gustav Lindenthal is 3 feet 3.6 inches at mid-span, the deflection of the stiffened eyebar suspension bridge submitted by the Canadian Bridge Company is 2 feet 9.58 inches at the center, and of the English Electric Company 1 foot 3.4 inches, my check figure on which gave 1 foot 6 inches, this suspension bridge being a remarkably rigid structure."

"The deflection of the cantilever bridge submitted by the Canadian Bridge Company under live load is 2 feet 1 inch. This is due to heat-treated eyebars being used largely in its design. The cantilever bridges submitted by the firms of Messrs. Dorman Long & Co., and Sir Wm. Arrol & Co., fabricated wholly of built-up plates and shapes, are much more rigid under full live load; in the former, the deflection would be 11.36 inches, whilst Sir Wm. Arrol & Co's cantilever bridge, built of special carbon steel plates and shapes, would have a deflection of 12.04 inches at center of the suspended span. The calculations for the official cantilever bridge using built-up members and eyebars gave a calculated center deflection of 1 foot 1-1/4 inches.

"The arch type of bridge is the more rigid; the three-hinged arch bridge submitted by Sir Wm. Arrol & Co. has a calculated deflection of 6.75 inches at the center, whilst the two-hinged official arch bridge tendered for by Messrs. Dorman Long & Co. has a deflection of 3.55 inches at the center under full live load. This tender provides for silicon and carbon steel built-up members throughout. My calculations made for a three-hinged arch bridge under full live load gave a center deflection of 7 inches, and for the two-hinged arch, the official design as submitted for tenders, 4 inches under full live load.

"It will thus be seen that a two-hinged arch bridge is more rigid than any of the other types, be it cantilever, suspension, or three-hinged arch. The two-hinged arch has only one-quarter the live load deflection that the most rigid suspension bridge yet designed has, one-third the deflection of a rigid cantilever bridge built up of plates and shaped, and *50 per cent of the deflection of a three-hinged arch bridge."

Distortion Under Unsymmetrical Live Load

"Inseparable from the question of rigidity is the question of distortion of the structure under an unsymmetrical live load. The symmetrical arrangement of the railway tracks on the bridge is a conspicuous feature of the layout of the structure as a whole, and its effect is to confine the forces due to live load on one side of the bridge

*Note discussion following.

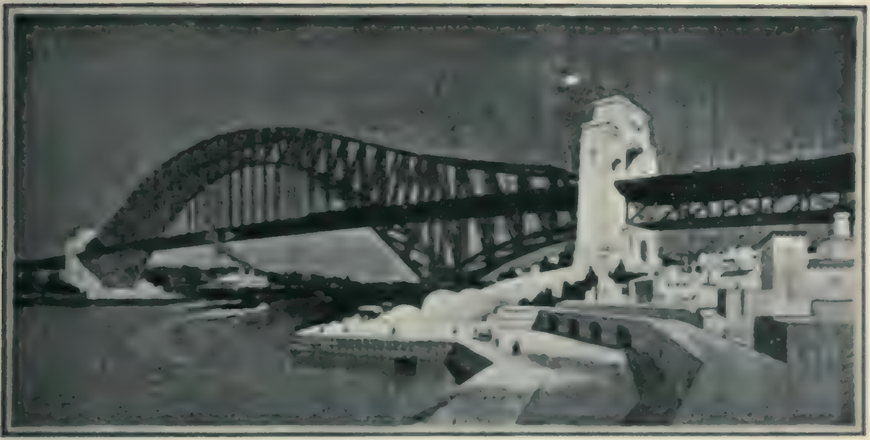
entirely to one truss, so that when the two tracks on one side are simultaneously loaded, one truss sustains maximum distortion and the other is unaffected. In a suspension bridge the deflections produced are of considerable magnitude and give rise to serious problems in the design of the lateral and transverse bracing. In a cantilever bridge and the very rigid suspension bridge submitted by the English Electric Company the deflections are much less than in a suspension bridge of ordinary type; but, nevertheless, there is sufficient distortion produced to render the use of lateral bracing on the main trusses impossible, and in a cantilever bridge to set up heavy twisting and shearing on the rigid sway frames, with consequent large secondary stresses.

"The same causes produce the effect of torsion on the suspended span of a cantilever bridge, when diagonally opposite portions of the channel span are loaded, causing the two corresponding supports of the suspended span to fall. The result is a twisting effect on the suspended span, which renders the use of the top lateral bracing on the span doubtful, and if the deflections are heavy, impossible. Consequently, also, sway frames of the suspended span are subject to heavy bending stresses.

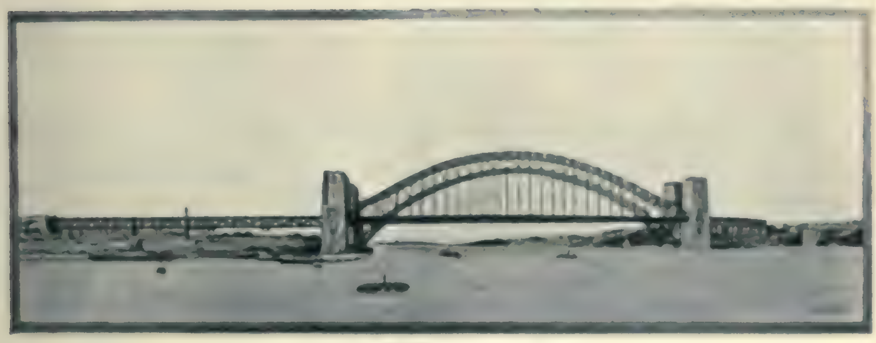
"In the case of an arch bridge, the tendency of one truss to distort under unsymmetrical loading can be restrained by means of lateral bracing of the same type as that required to resist lateral wind force, the relative distortion is reduced to a relatively small amount, and produces no special difficulty in the design of bracing, with very small secondary stresses in the main trusses."

"It is not an unlikely condition that the two railway tracks on one side of the bridge will be loaded simultaneously whilst the tracks on the other side are unloaded. The deck of the bridge would than have a lateral cross-fall under live load, governed in magnitude by the deflections of the various bridges under consideration, which could be immediately reversed in direction if the other pair of tracks were loaded. This tendency to see-saw under the action of the railway trains would produce a racking and straining of the rigid connections of the deck, which is more pronounced the less rigid the structure is. This see-sawing would be greater with a suspension bridge than with a cantilever bridge built up of plates and shapes, whilst with the arch bridge the deflection is so small that this tendency to dance would not be noticeable.

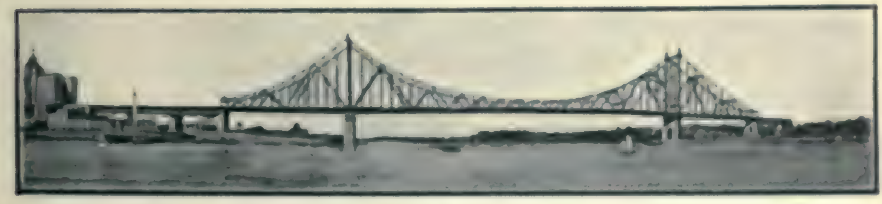
"The two-hinged arch bridge is least affected by unsymmetrical loading, i. e., when one truss takes the full railway load and the other is unloaded. It is, therefore, to be preferred to either the cantilever or the suspension bridge."



Cantilever Arch Design



Accepted Two-Hinged Arch



Braced Suspension

7. Approaches: The North approach of the bridge was on a curve, and for that reason the arch design which does not interfere with the easy curve was much to be preferred.

> "With all the suspension and cantilever bridges, the best curve which can be obtained on the bridge is 500 feet in radius. With an arch bridge a curve of 1,200 feet radius can be obtained, the reason being that the cantilever and suspension bridges have straight shore spans and the railway cannot begin to curve away from the line of the end spans until it passes the ends of the anchor arms or the anchorage of the suspension bridge, a distance of approximately 500 feet. With the arch bridge the railway can begin to curve away at a point 500 feet nearer the water. A saving in maintenance and running costs over either the cantilever or suspension type with a greater running speed can thus be attained with the arch type of structure."

8. The Breadth of Specification under which tenders were invited appears from the preceding quotations and clauses 8 and 9 governing the choice of steel and details of construction.

> "The cantilevers, girders of suspended span, arches, cross girders of main bridge, and girders of approach spans may be constructed wholly of carbon steel or silicon steel, or wholly of alloy steel, or partly of carbon steel or silicon steel and partly of alloy steel. The proportion of alloy steel to carbon steel is to be determined by Tenderer."
>
> "The details of construction of superstructure are left to Tenderers. The Bridge may be constructed wholly of built-up riveted members, or the tension members may be constructed of eyebars and pins."

But the preference for rigid construction is made evident under Clause 48 which required that the plans submitted must show all deflections of all parts of the main bridge and approach spans under maximum loading specified. As the writer interpreted this specification whereas the structural design with high working stresses for heat treated eyebars would be considered, so also would the deflections of the structure under its maximum load be duly considered and compared with other designs using lower working stresses and presenting correspondingly reduced deflections.

Every advantage was apparently given the North American tenderers. The heat treated eyebar was allowed 2½ times the working stress of ordinary carbon steel. Duty was modified in favor of the eyebar because there was no equipment in Australia for its manufacture. Yet, notwithstanding these concessions, the unit prices of the American bidder and the Canadian as well, were

67 to 85 percent higher than the successful English tender. Perhaps 16 percent of this might be chargeable to freight and duty, the balance to questions of exchange, excess profit and dependence on unreasonably high sub-figures for the masonry work. Price comparison is accordingly of little interest.*

The most striking divergence is that of the Belgian Australian bid of 10,700,000 pounds in rough numbers, for a cantilever suspension design against 4,220,000 pounds for the accepted arch design.

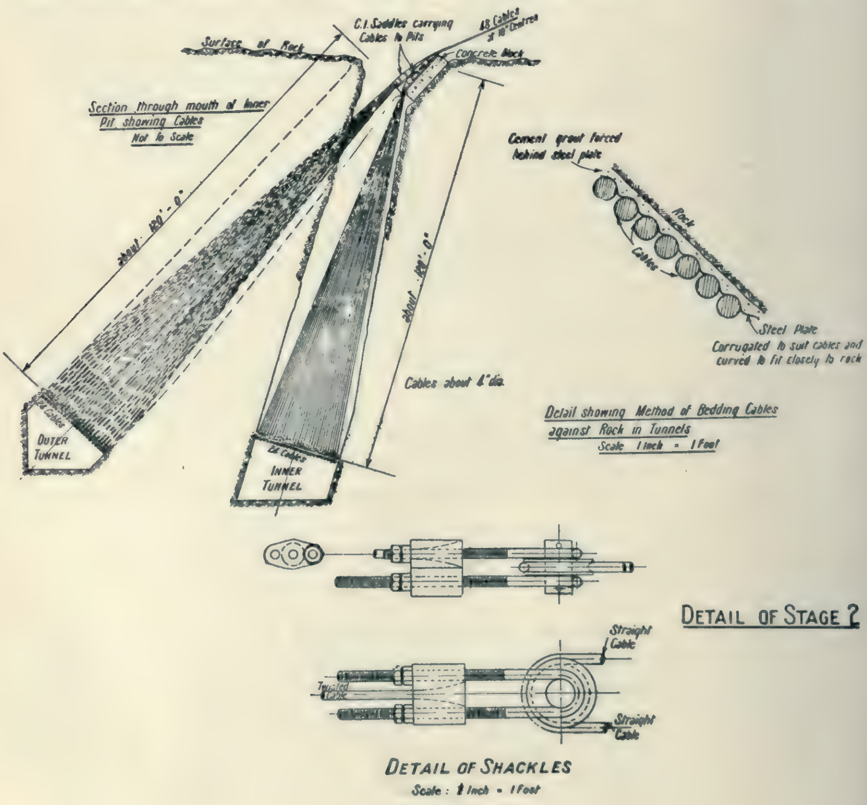
9. Lack of Economy of the Cantilever Suspension is indicated not only by price far out of line but also by comparative tonnages. The braced suspension design of the Canadian Bridge design weighed 38,000 tons. The cantilever suspension Straus type weighed 43,940 tons, or over 15 percent greater weight than the braced suspension type differing little in weight from the stiffened suspension of the English Electric Company designed by Mr. Steinman with 46,100 tons of metal and much greater rigidity.

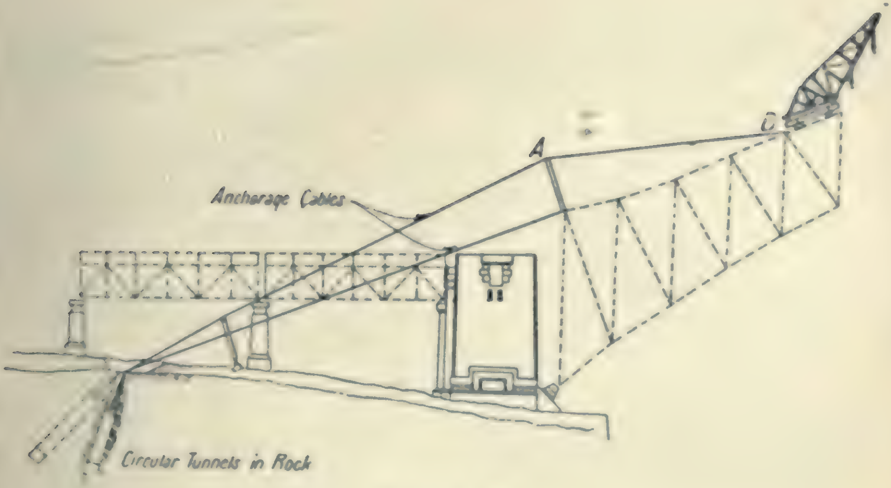
10. Proposed Erection Methods are of interest. The cantilever design of McClintic Marshall was to be erected by cantilevering out following the procedure with the Beaver Bridge, while the design of Sir Wm. Arrol & Company would follow a like method to that used for the Forth Bridge. The Canadian Bridge Company would float in the central span and jack it to position with equipment like that used at Quebec. With the braced eyebar suspension similar to the design which Mr. Lindenthal proposed for Quebec a string of eyebars would be placed and coupled from a light cable suspension; then the chain proper built up from that by heating the bars with steam instead of forcing them on the pins by hydraulic jacks as was proposed for the erection of the eyebar suspension of the preliminary design for the Manhattan Bridge afterwards discarded in favor of the stiffened cable suspension structure.

Elongating the bars an amount equal to the dead weight strain of the chain might be more advantageously effected by an electric heater or pad to be applied to the sides of the bars instead of attempting to heat them by steam.

The report on the tenders shows that the accepted arch design is to be cantilevered out from each side, restrained by back stays made up of 48 cables which enclose a mass of bed rock by passing through semi-circular tunnels driven 120 feet downward on an incline on each side and curved across to meet the tunnel on the other side. Corrugated plates curved to fit to the top of the cross tunnel backed by cement grout furnish the bearing for the cables. The tunnel shafts are to be approximately 10 feet by 6 feet by 140 feet apart transversely of the center line of the bridge. The cables passing through the tunnel are to be of twisted wire terminating above the ground level in U-bolt sockets for connection to the length of the back stay cable from the shackle at the ground line to the truss attachment above. The lengths of cables from the ground to the point of attachment to the truss are to be made of parallel wire strands carried round steel spools at each end of each strand. Adjustment of the lengths of back stay cable is to be effected by screwing up the shackles the condition of equality of stress to be verified by equal sag of the different strands or cables.

Driving 1,600 feet of tunnel through solid rock for back stay anchorages and fitting the top surface of the lower tunnel with heavy corrugated plates curved to match the curvature of the tunnel and backed by grout for bearing of the cable contrasts strongly with American practice in erecting the Niagara arch.


C.I. Saddles carrying Cables to Pits
18 Cables 3' 10" Centres
Surface of Rock
Concrete Block
Section through mouth of Inner Pit showing Cables Not to Scale
about 120'-0"
about 120'-0"
Cement grout forced behind steel plate
Rock
Cables
Steel Plate Corrugated to suit cables and curved to fit closely to rock
Cables about 4" dia.
Detail showing Method of Bedding Cables against Rock in Tunnels Scale 1 inch = 1 Foot
22 Cables
OUTER TUNNEL
22 Cables
INNER TUNNEL
DETAIL OF STAGE 2
Straight Cable
Twisted Cable
Straight Cable
DETAIL OF SHACKLES
Scale: 1 Inch = 1 Foot

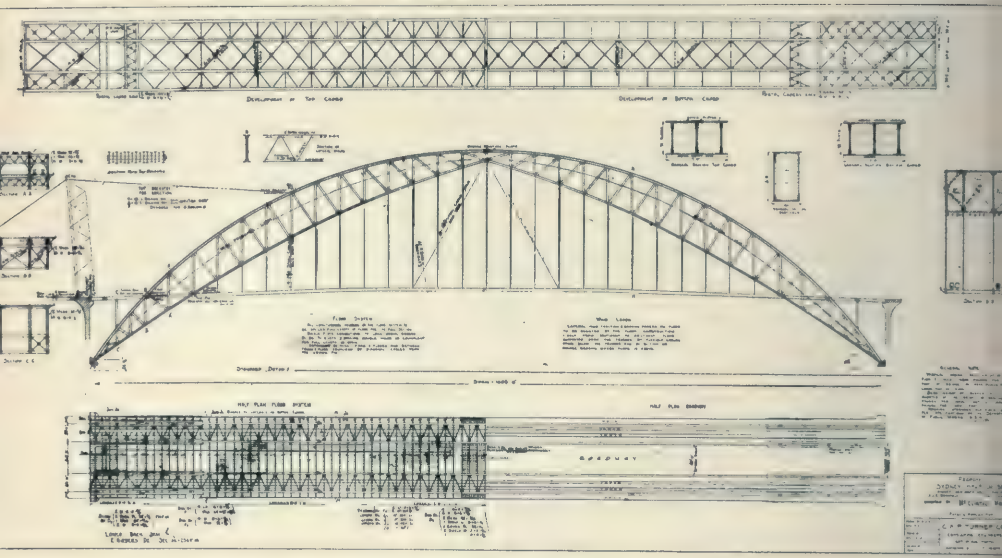


STAGE 3

11. The Frontispiece Three-Hinged Arch Design is the author's conception of the most desirable type of bridge frame in economic weight of metal, ease of erection and rigidity of finished structure for the conditions and span involved at Sydney.

Number of Trusses: The great width of the roadway caused the economic adoption of the four lines of main trusses for the approach span in the official design. The same reasoning applies with greater force to the channel span since by adopting four lines the back stay section may be cut in half by erecting and coupling up the center arch trusses first and then building out the side trusses supported by the center arches. In this way the creeper traveler may be reduced to common ordinary proportions, the heaviest member to handle is reduced to a maximum weight of sixty tons. Greater rigidity under the lateral forces of wind results from the greater out to out spacing and a maximum economy of deck material is attained.

Channel Span Trusses: Economic considerations require an approximately symmetrical disposition of the metal for the ribs about the mean pressure curve, which is substantially parabolic in form. A circular curve for the outline of top rib was adopted with a crown about twenty feet above the apex of this parabola. The main ribs at the quarter points are spread vertically 95 feet to resist cantilever erection stresses without overstrain or provision of excess material.



Elevation Floor and Bracing

The arch is designed to combine the desirable characteristics of the three hinged arch with the rigidity of the two hinged type by the introduction of a friction joint at the crown twenty feet above the center pin, made up of interlocking steel and bronze plates, clamped by the weight brought upon them by the hanger cables they support. When carrying the dead weight of the floor only, temperature change can produce at most a horizontal pull or thrust of fifty tons when the joint slips and temperature stresses are relieved. On the other hand under live load the increased stress on the cables locks the friction joint so securely that the truss then acts as a two hinged arch, thereby effecting a reduction in the maximum stresses of the arch ring. The truss thus obviates the vibration under live load of the three hinged arch without sacrificing its certainty and ease of analysis. It obviates the high temperature stresses of the two hinged type and the laborious and less certain computation thereof while securing its rigidity and economic advantage in supporting live loads.

Distribution of Load on the Main System: Dead load and wind load will be resisted by the trusses as an indeterminate system in accordance with the principle of rigidities. That is, the rigid sway system will tend to distribute dead and live load with approximate equality between the trusses under the condition of no lateral wind. When the outside trusses are strained by the wind load their vertical rigidity will be reduced below that of the inner trusses and the latter will then under wind carry more of the dead and live load than the outer trusses which oppose three times as much resistance as chord members to the wind load and accordingly five percent less than a quarter of the dead load is figured on the outside trusses under wind and a similar amount more than a quarter on the inner trusses which are but little strained under wind load.

Main Trusses of Approach are four in number but spaced so that each carries the same load, thus securing uniformity of details. They are figured for 5,000 pounds live load per foot with twenty percent impact—a somewhat heavier live load than specified. The consideration of utilizing floor material in erection influenced the arrangement of parts. The single system Warren type has top chords functioning as longitudinal girders carrying the floor from panel point to panel point. The excess section thus required to function as a floor girder is utilized as an erection or toggle strut in the erection of the channel span and the rolled joists are utilized in the same manner as those of the channel span where required.

Rigid connection of the buckle plate and brace angles to the joist and main trusses imparts to the deck enormous lateral rigidity and stiffness. Every part of the floor system functions as an element in a combination lateral truss or girder in addition to carrying its vertical loading so that wind and centrifugal stress in the truss chords is almost negligible.

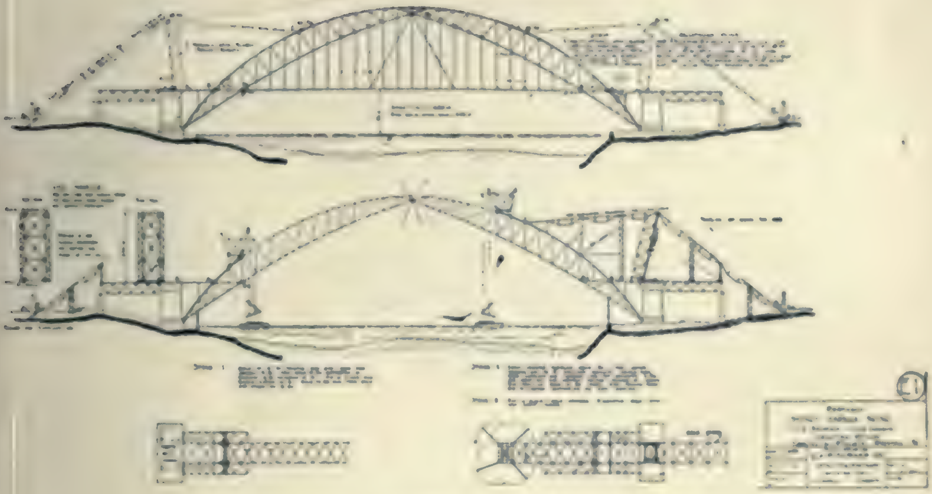
Floor Construction: To utilize the deck material in the dual manner just proposed, i. e., for erection of the main trusses in the first instance and as floor members in its final position, the rolled steel floor joist have been run longitudinally. Railway track stringers have been made up of twin rolled steel joist instead of riveted girders and all joists are furnished in from sixty-four to sixty-eight foot lengths fully spliced for making up the back stay for cantilever erection and their final function as deck joist. In like manner the distributing girders are run transversely instead of longitudinally and main girders longitudinally instead of transversely. The main girder sections of a truss panel length and like the joist are spliced continuously from support to support. This arrangement tends to distribute concentrated live loads upon the main truss and enables the main girders to function as wind chords of the floor system.

By this modification every member of the floor is made to act as an element of a combination horizontal or wind resisting truss from abutment to abutment, the buckle plates being rigidly connected to the main longitudinal girders and that they may function efficiently they are stiffened by 4 by 3's transversely of the roadway both on top and on bottom.

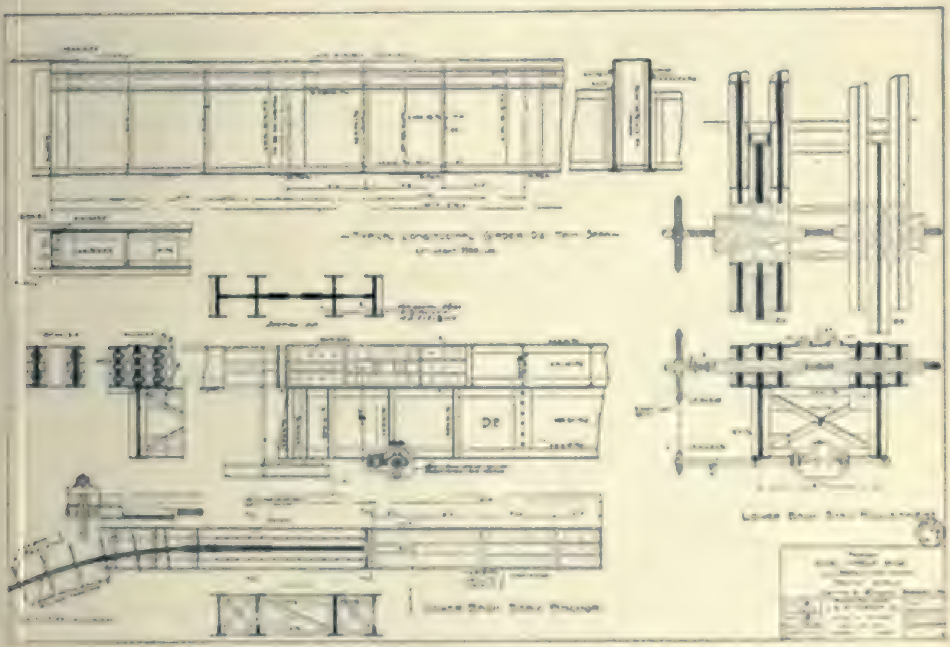
Tests recorded in Carnegie Hand Book show that by turning the buckle down so that the plate acts in tension, three times the strength is developed that is secured with the buckle turned up. The plates are accordingly turned down, made 5⁄16 inches thick with double Ts.

Cable Suspension of the Floor: Support of the floor from the main arch trusses is effected by galvanized plow steel suspension bridge cables. Minimum ultimate strength of the wires 200,000 pounds per square inch after galvanizing, elongation not less than four percent in eight inches with a maximum phosphor content .04, sulphur .03. The ultimate strength of these cables shall not be less than 283 tons each when tested in sockets similar to those designed for the service. Each longitudinal girder will be suspended by four of these cables at panel points with similar attachments above to the truss and below to the floor beam. The selection of cables in place of structural suspenders is for the reason that the cable exposes but a negligible surface to wind and is unaffected by bending stress, and because of its flexibility it transmits no secondary stresses to the supporting truss, thus permitting the floor and the truss to function independently in resisting the lateral force of wind.

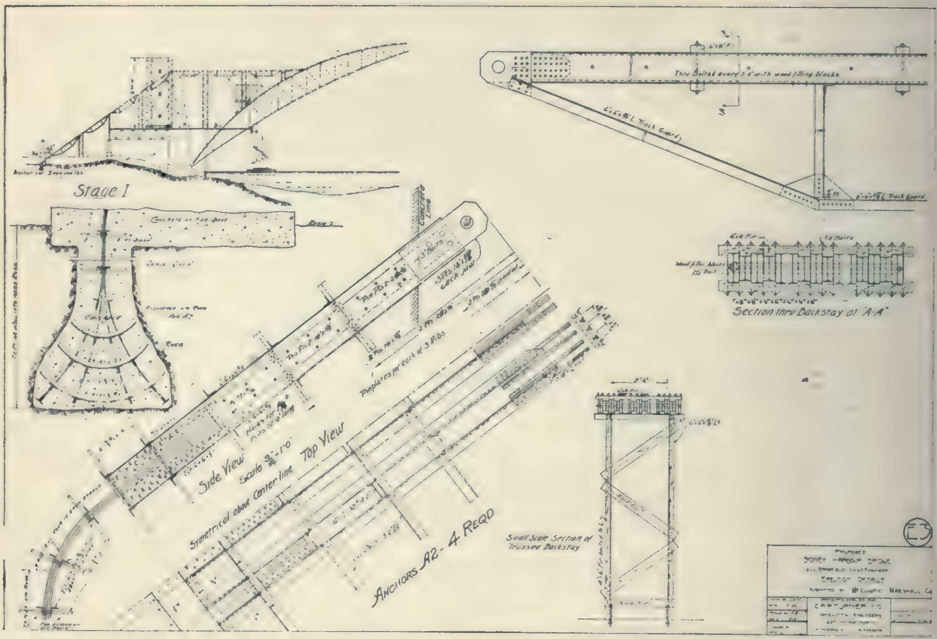
Safe Erection: Depends on the details of the supporting frame and its adjustment in building out the main trusses. The first step or stage in erection outlined in erection sheet E1, the building out of the skew back sections of the main arch which are intended to be supported on reinforced concrete columns with adjustable wedge tops, the material being raised by tackle from the floor of the pylon. After building out the lower sections the lower back stay girders will be placed in position and connected to the back stay anchorage buried in reinforced concrete pylon top designed to take a pull of five million pounds. The adjustment of these back stay members is to be made by horizontal screw wedges operated by fifty horse power motor through Ewart Link Belt Chains and sprockets. This method of driving permits the screw which operates the opposing wedges to move longitudinally of the bridge without interfering with the transmission means. The back stay girders D2 straddle, instead of cutting through the main ribs and are attached thereto by ten inch pins. The panel points of the main truss are arranged so that these pins come at a panel intersection.



Erection Method



Adjustment Lower Back Stay



Upper Back Stay Anchorage

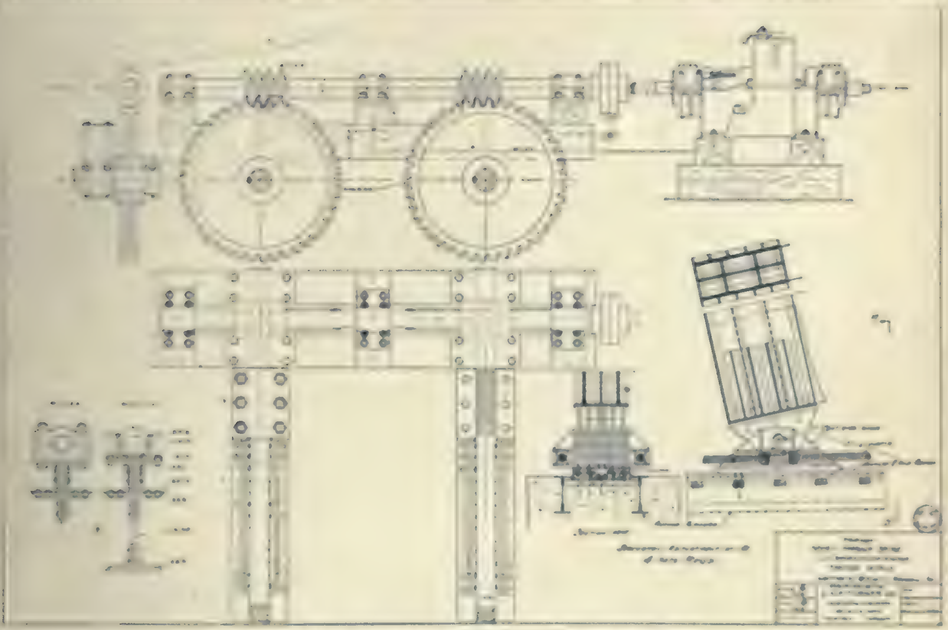
E-2 and E-3 shows the general scheme of erection in which the utilization of rolled steel joists with back stay ties is illustrated and the use of an approach truss as a pitman for the erection toggle is also outlined.

E-4 shows the wedge adjustment of the lower back stay and the thorough anchorage of that back stay in the mass of the pylon.

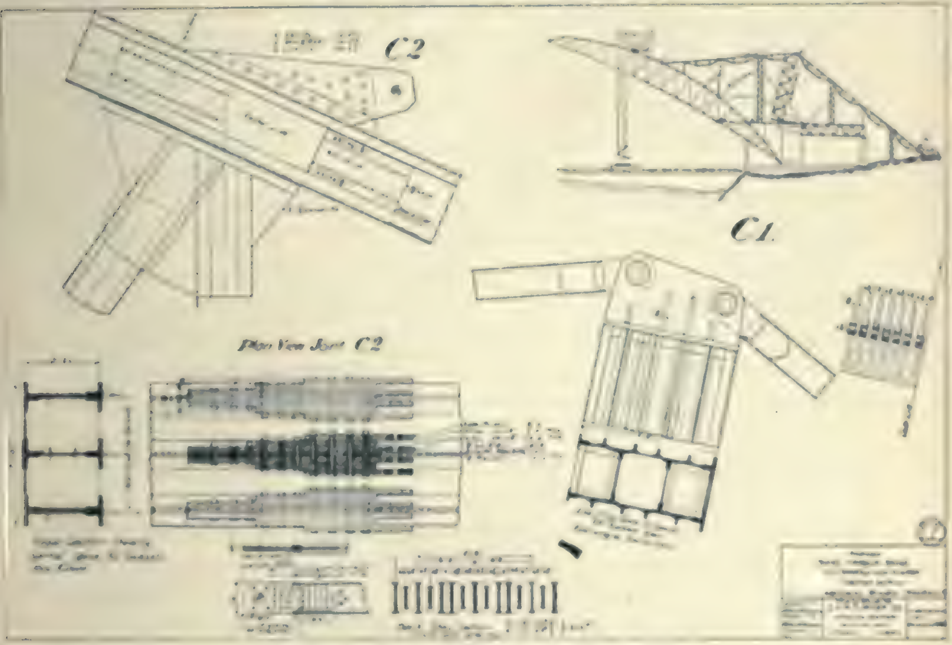
E-3 shows the detail of the sections of the back stay and the anchorage of the upper back stay down into the ledge under the second pier of the approach from the pylon both North and South.

In order to reduce the bending moment on the pins, subwebs are introduced between the girders D-2 so that the bearing on the pin connecting the back stay is provided on both sides of the main web of the truss chords thus permitting the entire pull of five million pounds to be taken up without severe bending stress on the pins at these joints.

After building out beyond joint C-1 the top back stay is to be connected by the rear booms of the creeper traveler. The top back stay is made up as before noted of the floor joists trussed by the track guard angles as shown in erection sheets 2 and 3. These back stays are pin connected at about 135 foot intervals so that they



Adjusting Device Upper Back Stay



Connection and Detail of Beam Stay

may accommodate themselves with negligible secondary stress to the shift of the toggle as the pitman truss is moved in or slacked away by the screws attached to the roller shoe at the lower end of the pitman which is operated by a fifty horse power electric motor of standard type through worm gear wheels having a diameter of forty-four inches.

The relative safety of erection operation depends partly on whether the weights to be handled are usual or customary, or unusual involving untried equipment.

The modification of the floor arrangement renders the heaviest member to handle fifty feet in length by eighteen tons in weight. The arrangement of four trusses in place of two reduces the thrust for the same load by half while the arrangement of the two ribs of the truss at the skew back so that they divide the thrust between them renders the rib member per panel length less than one-fourth the weight of that involved were there two trusses with but a single rib at the skew back.

The weights to be handled are thus reduced to ordinary amounts. The span of the traveler is reduced to but little over sixty feet (an ordinary span) and the erection equipment needed is not much heavier than that required for ordinary work.

In the adjusting mechanism simple slow motioned screw and wedge mechanisms motor driven are depended on throughout as will appear from the detail drawings submitted. Complication, uncertainty and risk are thus eliminated. The idea is to render the great task of erection an aggregation of operations, none of which embody more than commonplace procedure and require nothing more than ordinary tackle.

Computation of Arch Trusses follow the graphic method which possesses the advantage of self checking if executed with sufficient care so that the diagram closes the crown thrust having been correctly determined by moments. The diagrams are carried out on a convenient unit panel load basis so that any modification of the computed dead weight requires merely a change in the multiplier and not a new diagram. The dead and live loads are treated as though both are concentrated at the points of live load application for simplicity. This is justified because the maximum stress in the vertical web members occurs in erection and these members alone are affected by this deviation. The procedure is as follows: Disregard the clamp first and treat the frame as though it were an ordinary three hinged arch loaded on the right half of the span and then on the left half. For uniform load sum the stresses so obtained for preliminary computation and multiply by the coefficient ratio, and then combine the dead load stresses with the live load effects for the *half* span loads.

Effect of the Friction Joint is analzed by a unit force diagram for *either* pull or thrust at the friction joint, the same diagram answering for both, the signs of the stresses being changed. Treat the coefficient of friction of the bronze and steel as 15 percent of the load and compute the resulting effect for dead and live load and combine with the foregoing stresses. The advantage of this invention is that the amount of friction developed is under the designer's control. By increasing or decreasing the number of friction surfaces the temperature stresses under dead load may be rendered negligible and full advantage taken of the greater clamping effect of live load to secure the rigidity and economic characteristics of the two hinged type while avoiding the high temperature strains incident thereto.

The type of friction joint intended to prevent any vibration due to the three hinged nature of the truss, has been successfully employed in five arch spans over the St. Croix River 350 feet center to center, 180 feet from the deck to the river under the heaviest railroad loading, so that nothing untried in this expedient is here proposed, though the makeup of the details presents considerable novelty.

The Determination of the Sectional Areas of Members is simplified because temperature stresses are largely eliminated by the determinate nature of the three hinged arch under dead load and the arrangement of the floor in such wise that wind force upon the floor, brake force, longitudinal force, centrifugal force and temperature effects of the floor are taken care of by the floor itself and do not enter into the stress determination of the truss members.

The symmetrical distribution of the metal about the mean pressure curve (with web stresses of the small magnitude incident to such a frame) renders the thirty percent allowance for secondary stresses much greater than necessary to cover them.

The greatest secondary stress in the main arch frame would occur in the triangle next to the center pin because of the greater divergence of these members from the direction of the mean pressure curve and a simple computation of this triangle will substantiate the foregoing statement.

Track Guards: Inside and outside track guards for railroad bridge floors of 6x6x$^5/_8$ inches are commonly used on American roads. The double guard of this size angle has the advantage that it can be used in the makeup of false work, travelers and trussing of the back stays during erection and afterwards incorporated in the finished structure. While these double guards would weigh and cost more than the single angle guard of the official design, they become economically less expensive when their function during erection is considered and for that reason this modification in the steel guards is proposed on the double ground of economy and enhanced safety against derailment. If the practice of Messrs. Fowler and Baker on the Forth Bridge were followed the guard angles might be turned the other way around so that in case of derailment the wheel would roll along the angle flange without bumping and could be brought back by derailing blocks at the end of the span. This modification if desired would involve additional track bolts only and would leave the angle backed up by the planking.

Compression Members: Compression members are more seriously weakened by secondary stresses than are tension members. The theory of secondary stress assumes that there is no torsion on the member, whereas the failure of the riveted compression member is almost invariably caused by twisting out of line. The ordinary column formula disregards this dangerous kind of deformation or secondary effect of the primary stress.

As a solid cylindrical prism is twisted, the length of the prism is increased by the torsion through an area seven tenths of the diameter of the prism or cylinder. Hence, a solid cylindrical column fails by pure flexure and crushing only. Torsion does not enter into its collapse, since it cannot shorten by torsion or twist out from under its load. On the other hand, the hollow built up section, noncircular in form, tends to twist out from under its load or shorten by twisting, a phenomenon noted in many tests by the Bureau of Standards at Washington, D. C., of full size compression members such as Z-bar sections, channel and plate sections, and the like.

In his final design the author worked out *octagon sections of the special form illustrated in his typical 770 foot double track span, thus avoiding lacing and simplifying details. The eight angle section for the web members adopted in the web of the 770 foot simple span had not then been devised.

With such makeup the metal for the main trusses could have been rolled in Australia except wide connection plates. The weight

*This design was completed too late for a formal bid—weight 40,000 tons including erection anchors, carbon steel floor system and silicon steel in trusses.

including erection anchors and metal was under 40,000 tons, carbon steel only being figured on for the floor system and silicon steel for the main trusses.

12. The Engineer's Award was Warranted and Fully Justified from the Business Standpoint? A modest advantage of a third in weight of trusses with much greater lateral rigidity and substantially the same vertical rigidity and ease of erection could not offset unit prices eighty five percent greater than those of the successful tender.

Conditions, such as tariff, variation in exchange, convenient or advantageous shop location, divergent judgment as to labor conditions and costs are foreign to the engineering problem of economic design which must needs be limited to the quantity of material of a given grade and kind of work that may be required and the results obtained with it in point of load carrying capacity and rigidity.

13. Comparison of the Vertical Rigidity of the Two Hinged and the Three Hinged Arch in the report is somewhat ambiguous in that it fails to differentiate between the type and the effect of specific details. In the three hinged arch as designed by the Arrol Company the pin was placed in the lower chord which was parabolic in form. Such position does not give the most advantageous rise within the height of the frame. The pressure curve, because it follows the line of the lower boom is not opposed under uniform load proportionally by the metal of both booms with the result that the deflection with this type of such a three hinged arch will be much greater than one with the pin more scientifically placed. In addition, the higher working stresses selected add about ten percent to the deflection as compared with the stresses used for silicon steel in the accepted two hinged design. This is true because the modulus of elasticity remains substantially constant with the different grades of steel.

14. The Appearance of the Three Hinged Arch (See frontispiece) quoting the report,

> "is not in its favor. The outline of the arch ribs deepest at the quarter points though in conformity with statical principles does not add to its appearance and the defined crescent shape of each half arch does not produce a satisfactory optical explanation of the transference of the enormous stress from the crown of the arch to the abutment at the springings."

The truth of the old woman's explanation of the divergence in taste is here apparent. The thrust must come to the skew back. Whether a more satisfactory optical explanation be furnished by a framework extending vertically upward 180 feet away from the line of action of the pressure curve with framework which can carry no material portion of symmetrical load or the crescent shape arranged symmetrically with respect to the natural pressure curve presenting in perspective eight substantial ribs converging upon the springing point instead of two as shown in the frontispiece and the cut of the accepted design respectively is a question the perspectives should enable the reader to decide.

Contrast the statement that the crescent shape is not in favor of the appearance of the three hinged arch though in harmony with statical principles with the characterization of the two hinged arch as conforming to the principles of truth for "beauty is truth and truth is beauty." "The (two hinged) arch, graceful at the crown has depth where it is wanted and the rib beautiful in its strength and simplicity demonstrates clearly its purpose taking the eye down to the abutments on either side without camouflage or interruption."

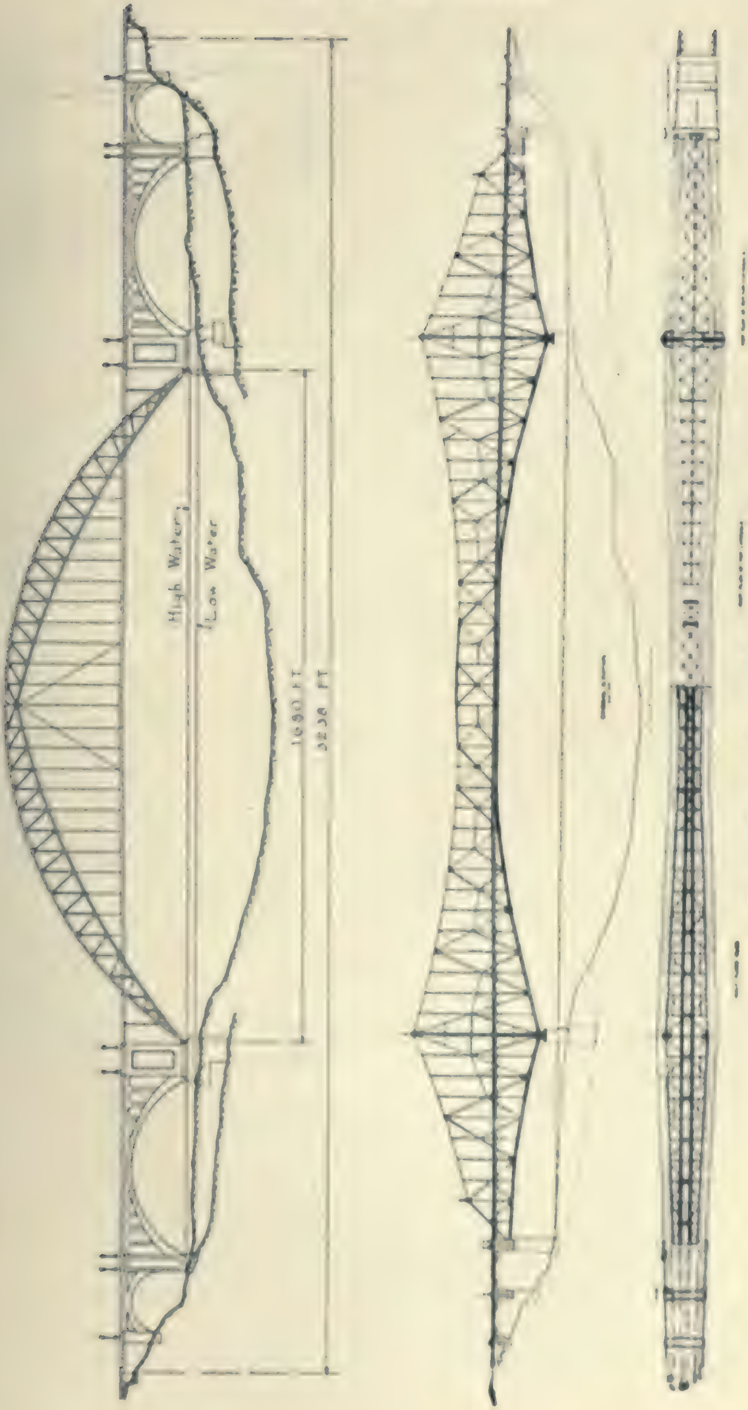
This language might well apply to a crescent shaped three hinged or two hinged arch but it is inapplicable to a two hinged arch frame work 180 feet deep over the pins where little depth is needed for moment.

What can be wrong with statical principles if they depart from the truth or what is beautiful in the departure from the truth if they are concededly correct?

The protuberant excresence of a hundred feet or so from the natural economic diagram for the loads to be finally supported is innocuous as an adjunct of the erection scheme when camouflaged as neatly as was done in the design of the Hell Gate arch by a false member apparently indicating the functioning of the haunch in part as cantilever instead of following the two hinged species. If we are to reason clearly in our analysis from the economic standpoint we must go behind such camouflage, examine the additional material required for the excrescent adjunct and compare it with the tonnage of the economic diagram which transfersits load directly to the support.

15. Contrast of Two Great Bridge Competitions, to wit, that for the Quebec Bridge and that for the Sydney Bridge is of interest, from the divergent viewpoint of the engineers in charge.

One of the Commission at Quebec considered that the structure was so gigantic in size that it would be inherently ugly anyway. Mr. Bradfield, Chief Engineer of the Sydney Bridge, on the contrary, considered the aesthetic appearance as an important attribute of the desirable structure and hence consciously or unconsciously worked toward economy.



Turner Design Proposed Quebec Bridge

In the general consideration of the problem at Quebec the economy of inclined reactions was given no consideration. The competition at Sydney avoided this short sighted practice with the result that they will undoubtedly have a structure three times as stiff vertically and much stiffer laterally than that at Quebec.

In discarding the foundations of the old structure at Quebec and building new piers no consideration was given to the possibility of using both the new and the old uniting them in a homogeneous whole in such wise that a large reduction in cost could be attained.

The writer worked up a design for an 1800 foot cantilever span for the Quebec location figuring it for the two railroad tracks, two highways and electric lines proposed by the Board of Engineers. This outline is illustrated in the accompanying figure and below it a supplemental outline in which concrete arches are disposed supported vertically on the old piers of the ill fated cantilever and furnishing an opposing thrust to an arch of the type illustrated in the frontispiece designed to fit the conditions of the Syndey location.

Comparing the two, the weight of steel would be 27,000 tons for the arch design against over double that amount for the cantilever. (See Engineering News 1912.)

The arch provides for liberal roadway for a trunk highway, opportunity for electric tram lines if desired and foot walks in addition to the railway tracks. Its deflection would be less than a third as great as that of the cantilever vertically and far more rigid laterally. The erection problem would be simpler and the total cost of the structure would approximate two thirds that of the cantilever design with improved appearance accompanying the reduction in cost and increased rigidity and highway capacity now lacking.

16. Antipodal Opinions Among Engineers Regarding Economic Design are indicated by the discussion of Paper No. 1496 Trans. Am. Soc. C. E., Co. LVXV 1922. Quoting Mr. D. B. Steinman—

> "In comparison with simple spans, the continuous bridge offers the advantage of economy of material which increases with the length of the span or with the ratio of dead load to live load. It is wrong to draw a general conclusion against the economy of continuous bridges based on a comparison of small spans (as Bender did in his book Continuous Bridges, 1876). It is difficult or impossible to arrive at reliable conclusions from abstract argument or theoretical considerations."

For purposes of comparison, Mr. Steinman states, he has made careful estimates of the weight of simple spans of 775 feet based on detail calculation of stresses and sections. The results are:

Weight of Trusses in Pounds per linear foot:

1 550-ft. Sciotoville Bridge	12 880 = 100%
775-ft. Simple span (Lindenthal's specifications)	15 300 = 119%
775-ft. Simple span (A. R. E. A. specifications)	17 650 = 137%

Total steelwork, in pounds per linear foot:

1 550-ft. Sciotoville Bridge	17 100 = 100%
775-ft. Simple span (Lindenthal's specifications)	20 300 = 119%
775-ft. Simple span (A. R. E. A. specifications)	23 000 = 134%

> "According to comparative studies made by Winkler, the saving for continuous bridges of two, three, and four spans is 16, 19, and 21%, respectively, when the span is about 300 feet, and 20, 24, and 28%, respectively, when the span length is about 500 feet."

As opposed to Mr. Steinman's calculations, the writer presents his design for two simple double track spans weighing 8,600 tons against the continuous two span structure covering the same opening weighing 13,240 tons, or apparently the continuous truss of two spans is one and one half times as heavy as two simple spans; and the author's design of the simple span is a little more than half as heavy as Mr. Steinman's design based on Mr. Lindenthal's specification and less than half as heavy as Mr. Steinman's design based on the A. R. E. A. specifications.

All this is easily accounted for when we consider the variation of the strain area coefficient with divergent triangulations represented on page 47.

Mr. Lindenthal points out that the steel weight per foot of bridge is 17,100 pounds as compared to the weight of 11,200 pounds per linear foot of the double track simple Ohio river span at Metropolis 720 feet center to center. Here, however, he attributes the light weight to the use of alloy steel eye bars instead of riveted tension members and the use of higher unit stresses.

In the author's design he has discarded eye bars because he finds that an eye bar chord increases the weight in a heavy design over and above that of the riveted chord. So here again the opinions of engineers of over thirty years' experience are antipodal upon fundamental questions of design. As in general engineering opinion is harmonious on purely mechanical questions and statical problems of computation their divergence on the question of design must be attributed to the failure of logical application of fundamental principles to the questions embodied in the economics of the problem the substantially correct solution of which measures the value of the work of the engineer from the commercial standpoint.

The fallacy of some of the arguments leading to conclusions regarding economic design is exposed in this discussion in the statement

> "If two spans of simple girders with a height of 1/5.5 show greater economy then there is no reason why greater height could not also be used for the same purpose in continuous girders."

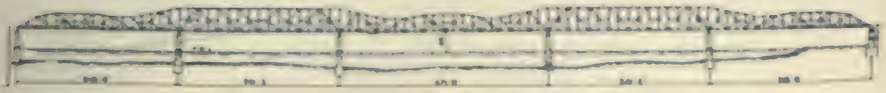
In the derivation of minimum strain areas it is found that the depth ratio is determined by the triangulation. We cannot say, if one truss is lighter, it is due to greater depth than that of another because if we increase the depth of the other we might increase its weight instead of reducing it, again indicating the present lack of scientific understanding of economic relations.

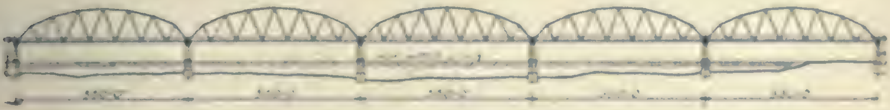
Antipodal view points again develop in respect to the bracing. The merit of the Sciotoville Bridge as viewed by Mr. Steinman is that the use of ordinary sway bracing of the rigid type was avoided on account of the high stresses which such bracing would suffer under one side loading of the bridge. Under the extreme case of one track of one span and the opposite track of the other span being fully loaded there would be a difference of two inches in the mid span deflections of the two opposite trusses. "To permit the unequal deflection without rupture the usual sway frames of rigid type were replaced by lattice frames combining strength with *elastic stiffness*."

Here is the disadvantage of the continuous truss which by the diagonal loading of opposite tracks on adjacent spans produces such large twisting tendencies that rigid bracing cannot be employed. The continuity effect is obviously most disadvantageous. No such difficulty is experienced in the fixed span which may be rigidly braced and because of its increased vertical depth presents far greater vertical rigidity and if the members are given sufficient bulk or breadth and rigidly braced the divergence in the loading of the two tracks will cause but slight torsional displacement in strong contrast to the known and admitted effect upon continuous and cantilever structures.

The Sciotoville Bridge is but a two span structure. As the relative weight of the continuous truss is claimed to decrease as the number of spans increase up to five or more, in fairness we should investigate the relative weight of a series of continuous spans in order to present the continuous truss in its most favorable light.

17. The Five Continuous Spans of the Thebes Bridge Permit a Direct Comparison with Simple Spans: The structure is built on the Gerber principle of hinging alternate spans





at points of inflection and is described in Engineering News of May 1905. The steel weight is recorded as 12,000 tons, or about 5 tons per lineal foot. The length centers of bearing of end span 2750 feet. It consists of five spans, four approximately 520 feet and a central one of 670 feet. It was designed for 5000 pounds uniform load per foot of track with a 50,000 pound concentrated additional load. The trusses were designed for 80 percent or 4000 pounds per foot for both tracks. The wind at 1000 pounds per lineal foot of bridge. From the writer's computation five simple spans all alike could readily be designed to provide for a 20 percent heavier load with less weight of metal and with greatly increased rigidity.

Summary: It appears that the arch is the most economic long span structure where the foundations are suitable. The cantilever arch ranks next, then the cantilever and the braced suspension. Heat treated alloy steel with the present equipment is not available for fabricated members of the arch while it may be utilized in eye bar tension members the braced suspension or inverted arch permitting longer spans with a narrow roadway than are possible to construct with the ordinary arch. Where navigation clearance is to be provided for the roadway 150 to 170 feet above the skew back the effective rise of the arch would be so much greater than the versed sign of the inverted arch that the advantage of heat treated alloy steel would be largely off set by the arch and a stiffer structure result, as was found to be the case in the Sydney competition. Continuous truss spans are not as economical as a series of simple spans with arched top chords nor are they as rigid. Where clearance requires shallow trusses in deck bridges continuous trusses with parallel chords may prove more economical than simple trusses. In like manner a series of continuous spans are advantageous in plate girder trestles in the stringers of the floor of the truss span and a material economy and increase in rigidity may be secured by their adoption.

APPENDIX

GENERAL SPECIFICATION FOR STEEL RAILROAD BRIDGES

By

C. A. P. TURNER, C. E.

1. The structures shall be made wholly of rolled steel except shoes, bases and machinery for draw spans which may be of cast steel.

2. **Types of Girders:** For independent spans up to 30 feet rolled beams in pairs with suitable separators may be used; for 25 up to 100 feet riveted plate girders; 90 to 120 feet lattice girders; rivet trusses for other and longer spans. In general thru double track bridges will have but two trusses to avoid spreading the tracks. Single intersection trusses will be preferred.

3. **Clearance,** overhead shall be 22 feet above the top of rails. Lateral clearance between trusses between the heights of 4 and 16 feet above the rail shall not be less than 16 feet. The clearance rectangle of 16 feet by 22 feet may be reduced by cutting from the corners triangular areas for braces such as gussets in thru girders having a base clearance at the top of the rail not less than 11 feet and at the top for knee braces not less than 8 feet.

Additional clearance in inches of 0.85 D on each side of the center of track and 1.7D between tracks will be required when the structure is on a curve, D being the degree of curvature. This additional clearance is intended to provide for a passenger coach 80 feet over all with trucks 57 foot centers.

4. **Spacing of Stringers** shall in general be 8 feet, center to center. Spacing of trusses shall be spread as the span lengths are increased for economy and stiffness in resisting lateral and overturning forces. In no case, however, shall the spacing of the trusses center to center be less than one twenty-fifth of the span length.

5. **For Determination of Dead Weight** of the structure for calculating strains, the timber shall be taken at 4½ pounds per foot B. M., and the weight of rails, spikes, and joints at 100 pounds per lineal foot of track.

6. **Loads:** The structure shall be figured to carry in addition to its own weight and that of the floor, the moving load for each track approximating Cooper E-60 loading as rectified in curves of uniform load, page 8. In computing for live load use the conventional method, the uniform load for the segment loaded being taken corresponding to the uniform load for moment for a span length equal to the length of the loaded segment. Provision for eccentricity of loading shall be made when the structure is on a curve.

7. **The Centrifugal Force of the Train** traveling at a speed commensurate with the curvature shall be allowed for as provided in Art. 10, page 15.

8. **Impact Effect** shall be considered and added to the maximum strains resulting from the above live loads as determined by the following formula:-

$$I = L\left(\frac{L}{D+L} \times \frac{140}{A+100}\right)$$

in which I is the impact strain, L the live load strain statically determined, D the dead load strain and A the length of the loaded segment which produces the maximum live load stress L. In case there is range of stress from L_t tension to L_c compression the formula becomes:-

$$I = \left(\frac{140}{A+100}\right) \times (L_c + L_t)$$

9. **Lateral Force of Wind** on the structure shall be treated as a uniform load of 200 pounds per lineal foot of span on both the top and bottom chord of truss bridges up to 200 ft. and moving wind pressure on the train at 300 lbs. per lineal foot. Allow twenty-five pounds per foot additional uniform wind load for each increase of 50 ft. in span above 200 feet. In the floor of the through bridge or deck bridge an additional shear on the laterals shall be provided for equal to the sum of that due to the uniform wind load plus one-tenth of the equivalent uniform load single track fully loaded. For the top chord laterals of the through bridge a shear in addition to that for the uniform wind load equal to three percent of the compressive stress in the end chord member for all panels to provide for the column action of the chords as latticed by this bracing.

10. **Bracing:** All lateral bracing shall be made of stiff members capable of taking tension and compression. The greatest slenderness ratio shall not exceed 110 L/R in any lateral strut.

11. **Trestles:** In addition to the dead live and wind strains before specified a lateral force of one hundred pounds for each vertical lineal foot of trestle bent shall be provided for or with no live load 500 pounds per longitudinal foot of girders acting at the center line of the girders in conjunction with the lateral wind force of 100 pounds per vertical foot of bent above specified.

12. **Longitudinal Bracing** shall be figured for the effect of suddenly stopping the train at any point considering the coefficient of friction of the wheels at 0.2.

13. **Variation of Temperature** shall be provided for by allowing one inch expansion for each 100 ft. of span length.

QUALITY OF STEEL

14. **Materials:** Rolled steel shall be made by the open hearth process and cast steel by the electric furnace conforming to the following analysis. (Electric furnace castings are specified as such castings cost no more and are free from the blow holes and defects common to the open hearth.)

	Shapes	Rivets	Castings
Carbon	As required for tensile strength		
Phosphorus, Maximum	.03	.03	.03
Sulphur, Maximum	.04	.04	.04
Silicon	to .35	.25 to .35	.25 to .35
Manganese, Maximum	.35	.35	.35
Chrome	.20	.20	....

15. **Ladle Analyses** shall be made by the manufacturer from a test ingot taken during the pouring of each melt, to determine the percentage of carbon, manganese, phosphorus and sulphur. A copy of this analysis shall be given to the purchaser or his representative.

16. **A Check Analysis** will be made by the purchaser from the metal at the center of the top of the ingot after discard in which case the tolerance for sulphur and phosphorus shall not exceed by more than twenty-five percent the above limits for ladle analysis.

17. **An Arbitrary Discard** from the top of the ingot of twenty-five percent of its length will be required to avoid excess segregation, and the addition of 6 oz. of aluminum per ton is recommended to reduce its amount to a minimum.

18. **Physical Properties:** The steel shall be uniform in character, have a clean smooth finish free from flaws. The tensile strength, *elastic limit and elongation shall be determined from samples cut from the finished material after rolling. At least two test pieces shall be taken from each melt and from each different shape of thickness varying by ¼-inch, one for tension and one for bending.

All samples must show uniform fine grained fractures of a metallic silver grey color free from fiery luster or blackish cast.

Tensile Properties	Structural	Rivet	Castings
*Desired Ultimate Strength, lbs., per sq. in.	78,000	68,000	78,000
†Elongation in 8 inches	1,600,000 / tens. str.	1,600,000 / tens. str.	1,200,000 / tens. str.
Elastic Limit in percent of ultimate strength	55%	55%	50%

*Permissible variation in ultimate strength, 4,000 lbs. per sq. in.

†For material over ¾-inch thick a deduction of 1 from the specified percentage will be allowed for each 1/8-inch increase over ¾-inches.

*For the purpose of these specifications, the Elastic Limit will be considered the least strain producing a visible permanent elongation in a length of 8 inches, as shown by scribe marks of a pair of finely pointed dividers.

If the yield point or drop of the beam can be calibrated for any machine and its speed to represent the elastic limit within 5 percent, it may be used for general cases. Test reports must state by which method the elastic limit was determined.

Bend Test: Samples of shapes 5/8-inch in thickness shall bend COLD in a roller bender 180° about a 2½-inch pin and straighten out again without sign of fracture on either side. Rivet rod shall bend 180° flat without sign of fracture.

Specimens of steel castings 5/8-inch thick shall bend 90° around a pin of 2 inches diameter.

Tension and bend test specimens from steel castings shall be taken cold from test bars attached to the castings, or from the sink heads where of sufficient size. All test bars shall be annealed with the castings.

19. **Variation in Weight** shall not exceed 2½ percent from that specified except in the case of plates:

(a) When ordered to weight for thickness 3/8 and over 2.5 percent over or under for widths under 100 inches and double this tolerance for plates wider than 100 inches.

(b) When ordered to gauge the thickness of each plate shall not vary more than .01 inch under that ordered.

20. **The Finished Material** shall be free from injurious seams, slivers, flaws and other defects and shall have a workmanlike finish. Plates 36 inches in width and under shall have rolled edges.

21. **Marking** with the name of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that each pin and roller may be stamped on the end. Rivet and lattice steel and other small pieces shall be shipped in securely fastened bundles, with the above marks legibly stamped on attached metal tags.

22. **Inspection:** (a) The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

(b) The inspector representing the purchaser shall have free entry, at all times while work on the contract of the purchaser is being performed, to all parts of the manufacturer's works which concern the manufacture of the material ordered. The manufacturer shall afford the inspector, free of cost, all reasonable facilities to satisfy him that the material is being furnished in accordance with these specifications. All tests and inspections shall be made at the place of manufacture prior to shipment, and shall be so conducted as not to interfere unnecessarily with the operation of the works.

23. **Final Rejection:** Material which, subsequent to the above tests at the mills and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop and shall be replaced by the manufacturer at his own cost.

PROPORTIONING OF PARTS

24. **The Basic Working Stress** in designing members of the metal above specified shall be 18,000 lbs. per square inch for the net section in tension bending block compression and bearing on pins and inside bearing on rivets. Shearing stress on shop rivets and webs of simply supported girders, 9,000 lbs. per square inch. For columns under detail limitations later outlined, the working stress P/A shall be U/2 in the formula

$$U = f / \{1 + nfL^2 / \pi^2 E r^2\}$$

in which f = 36,000 (twice the basic working stress, see Art. 10, Sec. IV); n shall be taken as ½ for angle channel and box sections and as 1/3 for octagon sections, because free from twisting. (The curves pages 13-14, Sec. IV, present a graphic solution for both cases.)

Members Subject to Reversal of Stress shall be designed to resist both kinds of stress safely without reference to the other except as provided in the impact formula.

25. **Combined Stresses, Axial and Bending,** due to dead weight shall not exceed the allowable axial stress. Bending due to dead weight of members continuous or fully spliced over panel points one-third the extreme fiber stress computed as a simple beam, will be added to axial stress. (See Flexure of Columns, page 8, Sec. IV).

26. **Combined Chord Stresses** due to wind dead, live, and impact stresses may exceed the basic stresses twenty-five percent, but the section shall not be less than required for dead, live, and impact stresses figured for the basic stresses allowed in Art. 24.

27. **Shearing** on field rivets shall be one-ninth less than allowed in Art. 24, one side bearing on shop or field rivets, 15,000 pounds.

28. **Bearing Pressures** on machined surfaces of sliding expansion shoes 1,000 lbs. per square inch. Bearing pressures on concrete masonry, 600 lbs. per square inch, on granite masonry, 800 lbs. per square inch.

29. **Plate Girders** shall be proportioned by considering as effective depth the distance between centers of gravity of the flanges and the effective flange area as that of the flange, plus one-eighth of the gross section of the web. The compression flange shall have the same gross section as the tension flange. The area of the web in square inches shall be such that the external shear force divided by the area does not exceed 9,000 lbs. for simply supported girders, nor 10,000 lbs. for continuous girders.

The flange section of continuous stringers may be six-tenths that for discontinuous stringers provided end panels are one-twelfth less in length than interior panels, the compression flange being milled where butting against the beam and spliced with a fifty percent additional area of plate and the tension flange spliced with twenty percent excess area over the beam. Compression flanges shall be substantially braced at intervals not exceeding 10 feet.

Web plates shall be stiffened at intervals about the depth of the girder apart from the quarter point to mid span with closer spacing toward the support. The maximum spacing shall preferably not exceed five and a half feet. Web plates shall be spliced for shear distortion, i. e., uniform riveting not less than three lines each side of the splice.

Stiffner angles shall be riveted to the web in pairs, milled and ground to fit snugley under the flanges and for intermediate stiffners shall not be less than two-thirds the width of the flange angle for simply supported girders, nor half the width of the outstanding flange for continuous girders. At points of bearing, the width shall be greater and the thickness such as to develop the proper compressive resistance.

Shear and bearing of rivets in plate girder flanges shall be figured to resist the moment reactions. (See Arts. 10-11, Chapter XXIV)

DETAILS OF CONSTRUCTION

30. **For Stiffness Depth Ratios** of L/D shall not under-run L/7 for discontinuous **stringers** nor one-eleventh for continuous stringers without reducing the basic bending stress in proportion to the reduction of those depth ratios.

31. **Depth Ratios of the Floor Beams** of the single track bridge for E60 loading shall not be less than L/4, nor less than L/5 for a double track span without reduction of the working stress pro rata.

For deck plate girders, 90 to 100 ft., L/10 to L/11 will be satisfactory in point of stiffness, while for spans intermediate between 30 and 90 depths, intermediate between L/7 and L/10 will be preferred.

32. **For Through Single Track Spans** the depth should not be less than L/4 for a two hundred foot span; L/5 for a three hundred and fifty foot span; L/6 for a five hundred foot span; L/7 for a six hundred foot span with a center to center of trusses, not less than 30′0″. For Deck Spans, depth ratios may be made less when combination floor stringers and top chords are used or where continuous parallel chord trusses are adopted but not under L/10.

33. **Panel Lengths** for main trusses may be as long as the section and slenderness ratios of the chords permits, but the lateral system panels shall not exceed one and a quarter times the distance of the trusses apart nor differ from an even division of the main truss panels.

34. **The Minimum Thickness of Metal** shall not be less than three-eighths inches except for fillers. Metal exposed to excessive corrosive action of smoke and steam in over head crossings shall be increased or protected by concrete covering.

The minimum thickness of gusset plates for main truss joints shall be nine-sixteenths inches.

35. **Connections** shall develop the section of the members regardless of the computed stresses and shall be made substantially symmetrical about the axis of the member.

36. **The Slenderness Ratio** for top chords shall preferably not exceed 65 L/R with a limit of 90 L/R for web members and 110 L/R for wind and sway bracing.

37. **Compression Members:** In built compression members the metal shall be concentrated in the flanges. When of box form, the lateral radius of gyration shall not be less than one and one-half times the vertical. The thicknesses of the web shall not be less than one-thirtieth the distance between the lines of rivets connecting it to the flanges when the flange consists of one angle or less than one-thirtieth of the distance between the edges of the flanges when the flange consists of two angles riveted to the web. The same requirements shall apply to the cover plate that applies to the vertical web but the one-thirtieth of the distance may be increased to one-forty-fifth where the web is stiffened by cross diaphragms at distances not exceeding 140 thicknesses of the metal and no latticing will be required providing the spacing of diaphragms is so close that the slenderness ratio L/R of the flange between diaphragms does not exceed 40.

All compression members who sections are connected by latticing only or by diaphragms, shall have tie plates as near the end as practicable of a length not less than the width of the member and a thickness unless stiffened, not less than one-fiftieth the distance between the rivets connecting them to the member. Members composed of two channels may be stayed by single lattice bars at 60 degrees to the axis, having a thickness not less than one-fortieth center to center of rivets, or by double lattice at 45 degrees connected by rivets at the intersection of a thickness not less than one-sixtieth the distance between rivets connecting them to the member and their width shall be

For 15-inch channels, 2½-inch, 7/8-inch rivets
For 10 and 12-inch channels, 2¼-inch, ¾-inch rivets
For 8 and 9-inch channels, 2-inch, 5/8-inch rivets

For larger sections, angle or shape lacing will be preferred to flats. Sections which are latticed four sides shall preferably be laced with angles or channel shapes at 60 degrees to the axis of the member. The riveting of the lacing for web members shall be capable of developing a shear equal to 3 percent total stress on the member. In case four angles are latticed together to form the compression member, the flanges shall be turned in rather than out to reduce the tendency of the section to twist. Where the length of the angle between lattice connections is more than 30 L. R., it shall be stayed with a cast brace or strap. Outstanding angle legs in compression shall be limited to eleven thicknesses for main members and thirteen thicknesses for secondary members.

38. **Diaphragms:** At all points where floor beams, portals or other bracing connect with the posts or chords, proper diaphragms must be inserted to distribute the loads and forces over the full section of these posts of chords.

39. **Joints** in compression chords and trestle posts shall be milled, spliced on all sides for at least eight-tenths of the compressive strain figured thereon developed by the rivets.

40. **All Tension Members** shall be stiff members with a vertical depth not less than one-thirtieth the distance between horizontal supports. The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane within a distance of four inches, which are on gage lines, one inch or more from those of the holes cut by the plan, the parts being determined by the formula

$$A(1—P/4)$$

in which A equals the area of the hole, P the distance in inches of the center of the hole from the plane.

In a pin connection at the shoe, the end section across the pin hole shall not be less than one and one-third times the net section elsewhere and the section back of the pin shall not be less than that of the body of the member.

41. **Details shall be Accessible** for inspection, cleaning and painting. Pockets which would hold water shall be provided with drain holes or filled with concrete.

42. **Expansion:** For girder spans of 50 feet and under ordinary sliding plates will be acceptable. For spans more than 50 feet, machine surfaces of the bearing plates shall be provided the top bearing plate to be scored for lubrication with hard grease and alemite gun connection for lubrication. Spans over 100 feet, the shoe shall be of cast steel provided with a pin connected to the truss, the bearing to be machined and scored for lubrication as specified for the shorter spans of 50 to 100 feet. The bearing shall be grooved to prevent lateral displacement and the finished surfaces given a coat of white lead and tallow before shipment from the shop.

43. **Pins** over 7-inches in diameter shall be forged and bored with a 2-inch hole along their axis.

44. **Rivets:** Minimum pitch of rivets shall not be less than three diameters. The edge distance 1½ diameters and end distance not less than 1¾ diameters. The pitch shall not exceed 6-inches in the direction of the strain or sixteen thicknesses of the thinnest plate connected. Field rivets for the floor and main truss connections shall not be less than 1 1/8-inch diameter. The minimum rivet pitch shall be used at the ends of all built compression members for a distance double their greatest transverse dimension. Unduly long rivets of more than four and a half diameters shall be avoided either by increased bulk dimension of the make up of the piece or use of larger diameter rivets.

45. **All Trusses and Plate Girders Shall be Cambered** so that under maximum live load and impact, the deflection below the horizontal shall not exceed one-fifth of the total deflection.

46. **All Spans Shall be Anchored to the Masonry,** for 50 feet and under by 1¼-inch bolts 2 feet long, for 50 to 100 feet, 1½-inch rag or foxtail bolts 2½-feet long set in the masonry with hot sulphur, for spans 100 feet and up 2-inch bolts set in the masonry 2½-feet.

SHOP WORK

The Workmanship shall be equal to the best general practice in modern bridge shops. Shearing and chipping shall be accurately done and all portions of the work exposed to view shall be neatly finished. Rolled material before being worked must be straight, gagging or straightening where necessary shall be done in a manner that will not seriously injure the material.

47. **Holes shall be drilled** in metal more than ¾-inches thick. Metal under ¾-inches thick may be sub-punched three-sixteenths smaller than the nominal diameter of the rivet and drilled to one-sixteenth larger size with a twist drill. Outside burrs shall be rymmered making a one-sixteenth inch fillet. Sub-punching shall be accurately done. Inaccurate or badly punched pieces may be rejected. Drilling whether through solid metal or sub-punched metal shall be done after the pieces forming the member are assembled and so firmly bolted together that the surfaces are in close contact. No interchange of drilled parts will be permitted when it is necessary to take the member apart for shipping.

48. **In Assembly for Riveting,** the parts shall be firmly bolted and pinned together before riveting is commenced. Surfaces in contact shall each be given two coats of shop paint before riveting.

49. **Stringer and Floor Beam Connections** may be sub-punched and drilled through a metal templet 1½-inch in thickness so that all such connections will be absolutely interchangeable.

50. **Field Connections** of main truss members shall be assembled and drilled in position. The parts thus assembled shall be match marked in the shop and diagrams showing such marks shall be furnished the engineer for inspection during erection.

51. **Riveting:** Rivets shall be heated uniformly to a light cherry red whether in shop or field and driven while hot. Loose, burned or otherwise defective rivets shall be replaced, care being taken not to injure the adjacent metal and if necessary they shall be drilled out. Calking and recupping will not be allowed. Shop rivets shall be driven by direct acting riveters which retain the pressure after the upsetting is complete. Field rivets shall be driven by a pneumatic hammer of ample weight for the size of rivet used. Rivet heads shall be full neatly made, concentric to the rivet hole and in full contact with the surface of the member.

52. Finished Members shall be straight and out of wind.

53. Floor beams, stringers or girders having end connection angles shall be milled to exact length after the connection angles are riveted on.

SHOP PAINTING

54. After the shop work has been inspected, the member shall be thoroughly cleaned of loose dirt and scale, oil, parafin and grease being removed with benzine or gasoline. It shall then be given a coat of approved shop paint applied uniformly and well worked into joints and open spaces. At the time of application of the paint, the steel must be free from moisture or frost and the coat shall not be applied in damp or freezing weather except under cover and then shall be kept under cover until the paint is dry.

55. **The Shop Paint Shall be Mixed** in the proportion of five pounds sublimed blue lead, three ounces of lamp black and two quarts of boiled linseed oil. Field paint shall preferably consist of two coats of polished aluminum paint mixed with the vehicle manufactured by the Tower Products Company, or equal. The paint shall contain two pounds of aluminum powder to one gallon of the vehicle.

SHOP INSPECTION

56. Facilities for the inspection of material and workmanship in the shop shall be accorded the inspector by the contractor and he shall be allowed free access to all operations and parts of the premises where the work is being conducted. Ample notice shall be given the engineer inspector of the commencement of work at the shop so that inspection may be provided. The inspector shall be furnished copies of all lists, shop detail drawings and instructions regarding the work to the shop by the manufacturer.

WEIGHING AND SHIPPING

57. **Payment for Pound Price Contracts** shall be based on the scale weight of the metal in the fabricated structure including field rivets shipped. The weight of field paint, boxing, barrels for packing and material used for staying members on the car shall be excluded. Any weight in excess of two percent above the computed weight will not be paid for by the purchaser. The computed weight shall be the theoretical weight of the shapes, rivet heads and plates up to 36 inches in width in the finished work, as shown on approved shop drawings. To the theoretical weight of plates above 36 inches shall be added half of the allowable over run of the Manufacturer's Specifications. In addition, ten percent for fillets and over run of castings will be computed.

58. Members weighing more than five tons shall have the weight marked thereon. Bolts and rivets of each length and diameter shall be packed separately. Also nuts, washers, pins and small parts. They shall be shipped in boxes, kegs or barrels, but the gross weight of any package shall not exceed 300 pounds. A list of the contained material and weight thereof shall be plainly marked on the outside of each package or barrel.

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Turner, Claude Allen Porter
Elasticity and strength
of materials in construction

Engineering

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